

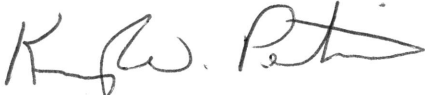


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INFORMATION AND SPECIAL INSTRUCTIONS: <p>Publication 293 (Geotechnical Engineering Manual), April 2018 Edition is to be issued with this letter.</p> <p>Chapter 15, Rock Slake Durability, of this manual was completed by subject matter experts and circulated for review and comments through the Clearance Transmittal process on CT# S-15-031. This release incorporates Chapter 15 into this edition of Publication 293.</p>		
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Geotechnical Engineering Manual

Publication 293
2018 Edition



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PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2018
GEOTECHNICAL ENGINEERING MANUAL

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PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2018 GEOTECHNICAL ENGINEERING MANUAL

BACKGROUND

Publication 293, “Geotechnical Engineering Manual” provides policy and guidance for geotechnical engineering activities for roadway and structure projects for the Pennsylvania Department of Transportation. This publication addresses geotechnical engineering activities required for three phases of projects, including: alignment alternatives analyses, preliminary design, and final design. Publication 293 is applicable to both design/bid/build projects and design/build projects, and applies to design done by consultants and design done in-house by the Department.

The geotechnical engineering work for structures must also follow the requirements specified in Publication 15 (Design Manual Part 4 (DM-4), Part A, Chapter 1, Sections 1.9.3 and 1.9.4), and Publication 448 (Innovative Bidding Toolkit, Chapter 3, Section 3.2.3.1). Additionally, Publication 293 provides policy and guidance on completing structure foundation tasks, and describes how to integrate geotechnical work for structures and roadway in the most efficient and coherent manner.

Since each project is unique and requires engineering judgment based on the specific project circumstances, this publication should not be used as the geotechnical scope of work for projects. Instead, the scope of work for each project should be based on project specific information and goals, using this publication only as a reference. Additionally, since roadway and structure projects involve numerous people from within the Department, outside agencies, and consultants, communication is critical from the onset of the project through construction to produce a successful project. This includes early (during project scoping) and continual involvement of geotechnical personnel.

The policies set forth in this publication are to be considered minimum requirements. Additional requirements may be added by the Department, or approved by the Department at the request of the geotechnical consultant, to fit specific project needs. The guidance provided in this publication is considered to be state of the practice; however, other acceptable means and methods to those presented in this publication exist and can be used, when approved by the Department. **The policy and guidance presented in this publication do not replace informed decision making, proper application of scientific data, or utilization of sound geotechnical engineering judgment and principles.**

Publication 293 is divided into numerous chapters. [Chapter 1](#) provides policy and guidelines for the geotechnical tasks and reports required to be completed for various phases of projects. The remaining chapters of Publication 293 provide policy and guidelines for

performing these geotechnical tasks and various analyses, as well as address a variety of technical issues and policy/guidance for some unusual and/or more challenging geotechnical conditions.

Equally effective management of construction is not possible if an equivalent awareness of the design principles leading to the construction proposed is not understood. As no one individual can realistically possess a knowledge of the details associated with all the components of either phase (design or construction), this is where early and frequent communication between personnel (between design disciplines and with construction specialists) is absolutely crucial. A lack of such communications may result in failures.

It is of value to adamantly affirm that while design and construction are separate functional phases, the two should never be separated in the practical sense. A functional and effective design cannot be achieved without constant consideration of constructability and a working appreciation of how an element is to be constructed.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

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GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 1 – GEOTECHNICAL WORK PROCESS, TASKS AND REPORTS

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1.1 INTRODUCTION

Chapter 1 of this publication provides policy and guidance with respect to:

- Project design phases
- Project complexity levels
- Geotechnical work processes
- Geotechnical scopes of work
- Geotechnical work tasks
- Geotechnical reports
- Geotechnical engineering for construction and maintenance activities

1.1.1 Geotechnical Design Phases

For the purposes of this publication, geotechnical design is divided into three phases, which include: Alternatives Analysis, Preliminary Design, and Final Design. These design phases are completed during Post Transportation Improvement Plan (POST-TIP) activities per PennDOT Publication 10 (DM-1). Based on guidelines in DM-1, Alternatives Analysis and Preliminary Design are completed during Step 6 – “Preliminary Engineering/NEPA Decision” of the PennDOT project delivery process and Final Design is completed during Step 7 – “Final Design and Construction”. Considerable geotechnical engineering will most likely be required in both Steps 6 and 7 for complex projects, whereas the majority of the geotechnical engineering can be completed during just one of these steps for less complex projects. Geotechnical engineering may be required during PRE-TIP and TIP activities. A brief summary of the three geotechnical design phases is given below.

1.1.1.1 Alternatives Analysis

This phase is needed for projects that are considering two or more roadway alignments and/or structure locations. These types of projects are typically for new alignments and are considered to be major (most complex). Geotechnical engineering is done during this phase to provide information to help select the preferred alternative and prepare the environmental document. Geotechnical engineering will also be needed in both the Preliminary and Final Design phases for these major types of projects. Where non-geotechnical issues (e.g., environmental, cultural resources, etc.) control the selection of the alignment, limited geotechnical engineering may be sufficient during the Alternatives Analysis phase. This phase is not needed for projects which are not considering multiple alignments, like routine improvements to existing highways, including reconstruction, widening and structure replacement.

1.1.1.2 Preliminary Design

This phase involves preliminary engineering investigations, designs and analyses to: support the environmental document when only a single alignment is being studied, provide input for preparation of the Design Field View Submission (i.e., 30% design), and to develop a detailed Scope of Work for final design. On major projects, considerable geotechnical engineering may be required to help set the preliminary line and grade, design preliminary geotechnical mitigation measures, and provide input for preliminary cost estimates. On less complex projects like structure replacements with minimal roadway work or minor roadway widening or realignment, a limited amount of geotechnical work may suffice, but it is preferred that as much of the geotechnical work as possible be performed during this phase.

1.1.1.3 Final Design

This phase is focused primarily on the preparation of the bidding documents, which consist of the final design plans and specifications. The level of geotechnical engineering required during Final Design will depend not only on the size and complexity of the project, but also on the amount of geotechnical engineering performed during the Preliminary Design Phase. Regardless of the size or complexity of the project, at a minimum geotechnical work will include: completing geotechnical tasks not done during Preliminary Design, and reviewing geotechnical related items, such as plans, details and specifications that are included in the contract documents.

1.1.1.4 Design/Build

It is highly desirable that all geotechnical investigations, necessary lab testing and geotechnical parameter selection be completed for inclusion in the Geotechnical Guidance Report, specifications, etc. Providing this information permits perspective bidders to develop a better quality and more cost-effective bid proposal.

1.1.2 Project Complexity Levels

Publication 10 (DM-1), Chapter 2, Section 2.1 divides projects into three complexity levels, including: most complex (major), moderately complex, and non-complex (minor). The amount of geotechnical engineering will generally be proportional to the complexity of the project. Typical projects with corresponding complexity level as indicated in DM-1 are described below.

1.1.2.1 Most Complex (Major)

Roadway projects that fall into this category are typically:

- New highways and interchanges
- Major relocations
- Major widening/capacity adding

- Major Reconstruction

Structure projects that are in this category include new, replacement, or rehabilitation of:

- Unusual bridges – segmental, cable stayed, major arches or trusses, steel box girders, movable, etc.
- Complex bridges – sharp skewed superstructure, non-conventional piers or abutments, horizontally curved girders, non-conventional piles or caisson foundations, etc.
- Bridges underlain by complex geology – mines, karst, etc.
- Anchored Retaining Walls/Tie Back Retaining Walls

1.1.2.2 Moderately Complex

Roadway projects that fall into this category typically include minor relocations and minor sections of new alignment. Structure projects that are in this category include bridges with footings on soil or rock with complex geology, friction or end bearing piles, bridges utilizing integral abutments, proprietary/non-proprietary walls, and noise walls.

1.1.2.3 Non-Complex (Minor)

Roadway projects that fall into this category are typically simple widenings and overlay projects. Structure projects in this category are typically replacements with minimal approach work, spread footings on rock or soil with non-complex geology, footings on conventional point bearing piles on competent rock, box culverts, sign structures (including Dynamic Message Signs), and noise and retaining walls designed using standards or computer software.

1.1.2.4 Complexity of Subsurface

The complexity or severity of the subsurface conditions will influence and possibly change the complexity of the project indicated in DM-1 from a geotechnical standpoint. For example, an overlay project is typically a non-complex (minor) project; however, if the roadway is distressed due to voids from limestone solutioning or deep mining and requires subsurface remediation, this project would be complex from a geotechnical standpoint. Conversely, a major reconstruction project is considered complex per DM-1, but this type of project may involve little geotechnical work if there is no alignment or grade changes, only complete pavement rehabilitation/reconstruction, and there are no unique/complex geotechnical issues.

1.1.3 Geotechnical Work Process

The Geotechnical Work Process, or order in which geotechnical tasks are completed, can and will vary for different projects. This work process is dependent upon the size and complexity of the project, the design contract agreement between PennDOT and consultant (if consultant design project), when approvals (e.g., Environmental Clearance, Design Field View, TS&L, etc.) are obtained, project schedule, and other factors. Certain geotechnical tasks have some flexibility in which phase of the project that they are performed. However, there is a general order in which geotechnical tasks should be completed in order to have the necessary

information required to effectively advance the project forward within the required project schedule, and to efficiently perform the required geotechnical work/engineering.

The typical work flow process for geotechnical engineering is graphically shown in [Figure 1.1.3-1](#). This work flow was developed for a major/complex project. The work flow process in [Figure 1.1.3-1](#) is initiated during the Alternatives Analysis Phase, and then is followed with Preliminary Design and Final Design Phases. This geotechnical work flow process also shows how structure foundation geotechnical engineering is incorporated into the roadway geotechnical engineering. This work flow process figure also applies to less complex projects by disregarding the phase(s) that is/are not necessary. For example, if the alignment is set at the start of the project then the Alternatives Analysis Phase is not necessary, and the figure can be “entered” at the Preliminary Design Phase. As previously indicated, there is some flexibility in when certain tasks can be completed and to what level of detail.

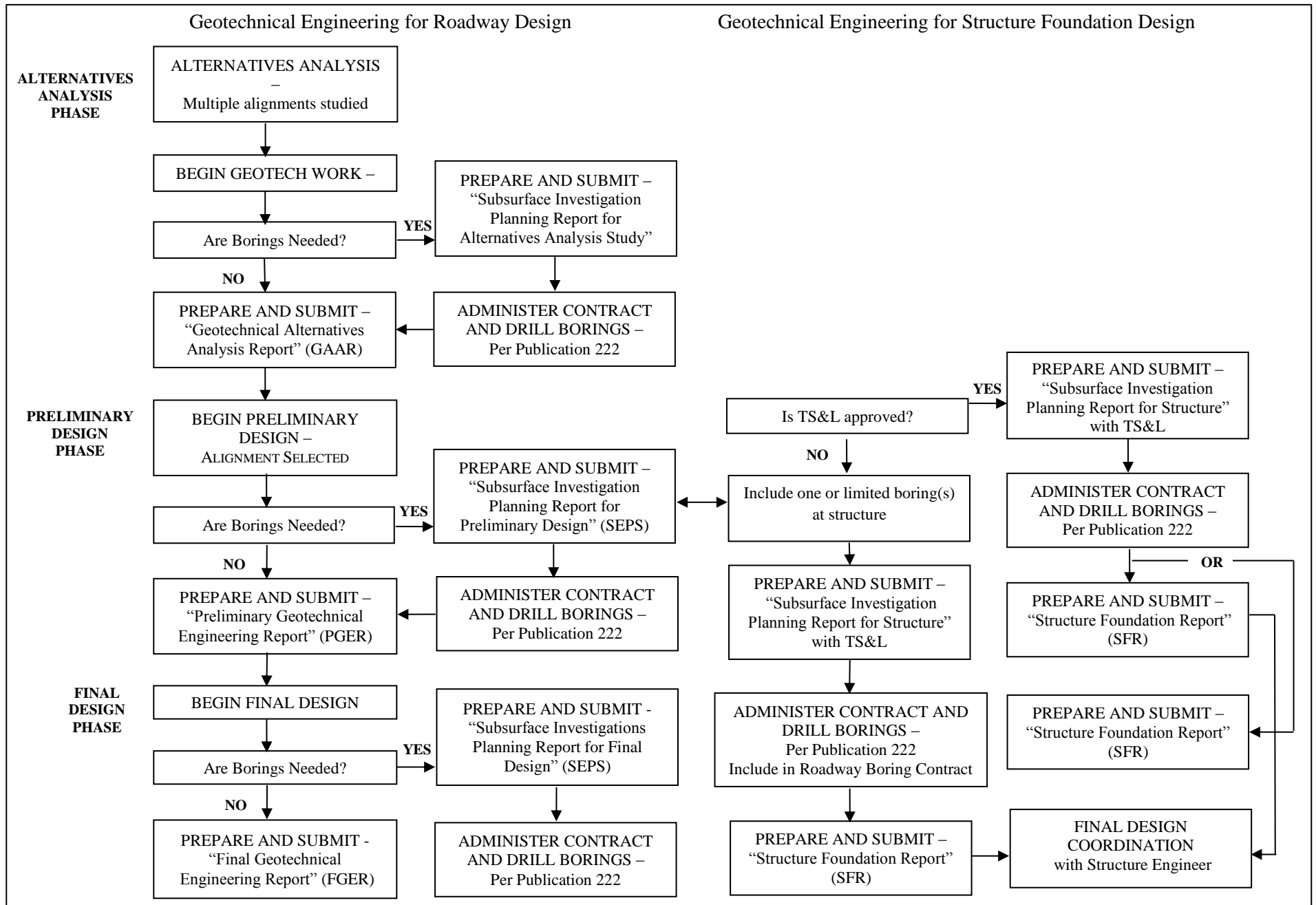


Figure 1.1.3-1 – Geotechnical Engineering Work Process

1.1.4 Geotechnical Scope of Work

A key factor in successfully, thoroughly and efficiently completing any work, including geotechnical engineering, is to develop the appropriate scope of work to guide the work and estimate the effort (time and cost) required to complete the work. A geotechnical scope of work must be developed regardless of whether the work is being done in-house by the Department or by a consultant. A well-defined geotechnical scope of work that is tailored to the individual needs of the project and specific project phase, lays the groundwork for a methodical and efficient approach to the geotechnical design activities, and also provides more accurate budgeting of time and costs.

The PennDOT District Geotechnical Engineer (DGE) must be involved from the onset of projects with geotechnical components, including participation in all scoping field views and in Pro-team meetings to develop the scope of work for geotechnical activities with the PennDOT Project Manager (PPM). It is understood that geotechnical scope changes during a project may occur due to unforeseen conditions and ongoing modifications made throughout the project design; however, scope changes are less likely when the initial scope of work is developed with project specific conditions and goals in mind.

As with any type of engineering work, the scope of work will depend on the type and complexity of the project. Major projects, including alignment alternatives analysis, new roadway alignments, and major roadway widening, will require significant geotechnical effort. Minor projects, including most reconstruction projects and small bridge replacements, will typically require less geotechnical effort. In addition to the project type/complexity, the geotechnical engineering scope of work will also be heavily influenced by the subsurface/geologic conditions. Projects with adverse subsurface conditions will require more geotechnical effort compared to those with favorable subsurface conditions. Regardless of the size, complexity and subsurface conditions of a project, the basic tasks required to complete the geotechnical engineering will be the same, but the level of effort required to complete the tasks will vary.

It is not sufficient to simply state in the geotechnical scope of work that tasks will be performed in accordance with Publication 293. That is not the intent of this publication. Be specific in the scope of work so the level of geotechnical effort is clear to all involved. Being specific will open dialogue among District personnel, or the District and consultant, and is necessary to estimate the time and costs associated with the geotechnical work. Provide specific discussion in the scope of work that includes, but is not limited to:

- Geotechnical information that is, or is expected to be, available
- Reconnaissance level of effort
- Extent of subsurface and laboratory testing programs
- Known or potential geotechnical issues and how they will be addressed
- Anticipated engineering analyses required
- Report(s) that will be prepared
- Geotechnical related contract documents that will be developed
- Anticipated number of progress/review meetings, location and attendees

- Higher than usual anticipated levels of efforts due to anticipated difficult or unfavorable subsurface conditions, or other complexities
- Potential construction phase services.

Typical tasks included in the scope of work are shown in the following section, and a detailed discussion of scope of work for the various design phases is provided in Sections [1.2](#), [1.3](#) and [1.4](#).

1.1.5 Geotechnical Work Tasks

Numerous geotechnical tasks are required to successfully and thoroughly complete a roadway and/or structure project. These basic geotechnical tasks include:

1. Reviewing scope of project, including any conceptual plans, profiles, etc.
2. Preparing geotechnical scope of work
3. Researching and reviewing available topographic and geotechnical information
4. Performing field reconnaissance
5. Planning, implementing, and supervising subsurface exploration program
6. Planning and implementing laboratory testing program
7. Interpreting subsurface information, laboratory test results and instrumentation data
8. Performing geotechnical engineering analyses
9. Providing geotechnical roadway and structure design recommendations and construction considerations, special provisions and details
10. Preparing geotechnical reports
11. Reviewing proposed plans, profiles, sections, etc.
12. Preparing geotechnical related contract documents (plans, cross-sections, details, specifications, boring logs, etc.)
13. Reviewing final contract documents
14. Provide consultation during construction, including geotechnical inspection, monitoring, and oversight, as appropriate.

Some, if not all, of these tasks may have to be completed during more than one phase of a project depending upon the size, complexity and subsurface conditions. For example, a complex project may require subsurface explorations during all phases to:

- Aid in selection of the preferred alignment in the Alternatives Analysis Phase
- Perform geotechnical design in the Preliminary Design Phase
- Refine the geotechnical design during Final Design Phase.

Alternatively, geotechnical tasks may only have to be completed during one phase of the project for some projects. For example, a subsurface exploration performed only during the Final Design Phase may be adequate for a small structure replacement or small roadway widening.

It is important that tasks which are performed during multiple phases of a project be done to gather new information, supplement previously obtained information or advance the

geotechnical design to the next phase. Tasks should not be duplicated during a latter phase unless necessary for further study. In planning and executing these tasks three critical factors must always be at the forefront of consideration:

1. There is no standard level of effort for geotechnical work – the level of effort is always site specific and highly dependent upon the nature and complexity of the subsurface conditions.
2. Every task undertaken must be purpose driven with the results serving the next phase of the geotechnical effort.
3. The geotechnical effort must be coordinated with all other aspects and disciplines of project development.

In general, always consider the implications of information obtained and observations recorded, relative to the next step of the process. Always be looking ahead. Detailed descriptions of these tasks and the level to which they are typically performed in the various phases are provided in Sections [1.2](#), [1.3](#) and [1.4](#).

1.1.6 Project Kick-off Meeting

The start of all geotechnical work in all phases of a project shall begin with a kick-off meeting. The purpose of the meeting is to discuss the geotechnical scope of work for the project, the geotechnical services (subsurface exploration, laboratory testing, etc.) that will be provided, and the deliverables that will be prepared and submitted. At a minimum, the Project Geotechnical Manager (PGM – PE or PG licensed in the Commonwealth of PA) and the District Geotechnical Engineer (DGE) will attend. It is important that project managers for PennDOT and the consultant (if used) be involved in order to convey the most up to date information, project constraints, design schedule, etc., and that other disciplines (highway, bridge, environmental designer, etc.) be consulted. Additionally, if requested by the DGE, a geotechnical engineer from the Bureau of Project Delivery, Innovation and Support Services Division, should participate. For small projects, and at the discretion of the DGE, this meeting may be replaced by a conference call, video conference, or electronic correspondence. Minutes of this and other meetings must be prepared by the PGM and submitted to the DGE within five (5) working days.

Prior to holding the meeting, available plans, profiles, sections, etc. should be reviewed to gain a general understanding of the project. In addition, it is helpful to have some knowledge regarding the soils and geology of the project area in order to help anticipate the complexity of the geotechnical aspects of the project. This knowledge can be obtained from published/available information, from project specific information obtained from previous studies, from past projects in reasonable proximity to the project, or from information from projects in similar geologic settings.

1.1.7 Geotechnical Submissions/Reports

There are six (6) types of geotechnical submissions/reports that are prepared for Department projects. Similar to the geotechnical tasks, the type of submission(s) required for a

project will vary depending upon the type, size and complexity of the project. The six types of geotechnical submissions/reports are:

1. [Subsurface Exploration Planning Submission](#) (SEPS)
2. [Geotechnical Alternatives Analysis Report](#) (GAAR)
3. [Preliminary Geotechnical Engineering Report](#) (PGER)
4. [Final Geotechnical Engineering Report](#) (GER)
5. [Structure Foundation Report](#) (SFR)
6. [Foundation Design Guidance Report](#) (FDGR)

A **Subsurface Exploration Planning Submission (SEPS)** must be prepared and submitted when a subsurface exploration (borings, test pits, geophysics, etc.) is proposed. This type of submission may be required to be prepared and submitted at various phases on large, complex projects. For example, a Subsurface Exploration Planning Submission will be required during the Alternatives Analysis, Preliminary Design, and Final Design Phases of a project if borings are proposed during each of these phases. The SEPS must include any reconnaissance work performed. This work provides a major portion of the justification for the proposed subsurface exploration plan. The SEPS would take the place of the Reconnaissance Soils and Geological Report (RSGER). In effect, the SEPS includes, combines, and replaces the RSGER and Problem Statement and Draft Exploration Plan (PSDEP) in most situations. Additionally, a submission may be required for each structure where borings are proposed for foundation design. When multiple submissions are required for a project, information gathered during previous studies/investigations should be used to prepare subsequent submissions. This will eliminate the duplication of work and will help to efficiently prepare the submissions. When multiple structures are involved, it may be more efficient to include two or more structures per submission. This should be addressed in the scope of work.

A **Geotechnical Alternatives Analysis Report (GAAR)** is required when alternative alignments are being studied and geotechnical input is needed to select the preferred alignment and prepare the environmental document. This report documents the geotechnical information gathered, and summarizes geotechnical analyses (if performed), recommendations and conclusions made during the Alternative Analysis Phase of the project.

A **Preliminary Geotechnical Engineering Report (PGER)** documents the geotechnical work that was performed through the Preliminary Design Phase of the project. Pertinent information from the Geotechnical Alternatives Analysis Report, if prepared, is included in the Preliminary Geotechnical Engineering Report. On major projects this report will provide recommendations to help prepare the Design Field View Submission. These recommendations are needed to: set the preliminary line and grade, design preliminary geotechnical mitigation measures, provide input for preliminary cost estimates, and discuss further geotechnical studies needed during the Final Design Phase. For other than single structure replacements, the components of the RSGER are performed as a part of the preliminary engineering studies (see Design Manual Part 1, Chapter 7, Section 7.1). For single span structure replacements and similar small projects, the reconnaissance may be conducted during the hydraulic study. In any situation where subsurface exploration (borings) is conducted for preliminary design, the

components of the RSGER would be included with the proposed subsurface exploration plan (SEPS).

A **Final Geotechnical Engineering Report (GER)** documents the geotechnical work that was performed during the Final Design Phase of the project. If significant geotechnical work (subsurface exploration, analyses, recommendations, etc.) was performed during the Final Design Phase, the Final Geotechnical Engineering Report must be a comprehensive document including all geotechnical information and recommendations from both the Preliminary and Final Design Phases. If only minimal geotechnical work was performed during Final Design, the Final Geotechnical Engineering Report may consist of an addendum to the Preliminary Geotechnical Engineering Report, or may be submitted in lieu of the Preliminary Geotechnical Engineering Report. If most investigation and analyses are performed during the Preliminary Design Phase, then the primary product of the Final Geotechnical Engineering Report will be final recommendations, construction special provision and construction details.

A **Structure Foundation Report (SFR)** is required for each structure (i.e., S- designated structure) on a project. The work to prepare this report is initiated after Type, Size and Location (TS&L) approval for bridges and culverts. This report is submitted with the TS&L for retaining walls and noise walls. Additional requirements for this report are included in DM-4, Part 1, Section 1.9.4.

A **Foundation Design Guidance Report (FDGR)** is required for Design/Build projects where an “As-Designed” foundation is not provided and the Design/Build team (contractor and engineer) is responsible for preparing the Structure Foundation Report. The purpose of this report is to provide the results of any subsurface exploration and recommendations of permissible foundation types to the Design/Build team. Additional requirements for this type of report are included in Publication 448, Chapter 3, Section 3.2.3.1. A similar report can be prepared for the geotechnical roadway design elements of a Design/Build project.

A detailed discussion of these reports, including content and format, is provided in [Section 1.5](#). Note that each of these reports is not just a compilation of geotechnical information. Instead, the geotechnical data is used to produce specific products necessary to fulfill a phase of or portion of a project supporting all other elements and disciplines involved in developing a complete project.

1.2 GEOTECHNICAL ENGINEERING - ALTERNATIVES ANALYSIS PHASE

1.2.1 Introduction

The geotechnical design process for projects that include the study of several (two or more) roadway alignments will commence with an Alternatives Analysis Phase. Geotechnical engineering will be required during this phase, and the level of effort required will be dependent on the geotechnical complexity of the project and the nature of the alternatives to be assessed. If alternative alignments are not being considered for a project, the geotechnical work will commence with the Preliminary Design Phase, which is discussed in [Section 1.3](#). The

Alternatives Analysis Phase is performed during Step 6 of the PennDOT Project Delivery Process.

1.2.2 Purpose

The purpose of geotechnical engineering during the Alternatives Analysis Phase of the design process is to provide information to aid in the selection of the preferred alignment, which includes information necessary to prepare the environmental document. Additionally, information obtained during this phase will help determine the level of geotechnical effort required during the subsequent design phase (i.e., Preliminary Design Phase).

It is important that geotechnical factors, both unfavorable and favorable, be identified during the Alternatives Analysis Phase so that they are accounted for in the alignment selection process. For most projects factors including environmental, cultural, and economical will be the main consideration(s) in the alignment selection, and geotechnical issues may be of little or no consequence. However, even for these cases, some level of geotechnical engineering need must still be considered. In such cases, it is important to ensure that the scope of geotechnical engineering investigation is appropriate to avoid encountering geotechnical conditions that can adversely affect the project. Additionally, geotechnical information is needed by other disciplines to better understand impacts of the various alignments, make informed and effective decisions for alignment, and assist with scoping for the next phase.

Adverse geotechnical issues may have an impact on the selection of the preferred alignment. Some of these include:

- Deep or surface mines
- Karst topography/carbonate (limestone) geology
- Acid bearing rock (ABR)
- Soft/compressible soils
- Talus/colluvium slopes
- Landslide prone areas/unstable soils
- Undesirable or potentially difficult/costly structure foundation conditions
- High water bearing zones or confined artesian aquifers
- Soluble rock such as claystone

From a geotechnical standpoint, alignments that avoid or minimize these adverse or unfavorable issues are preferred. However, avoidance of adverse geotechnical conditions is not always an option when selecting the preferred alignment because of other issues, such as environmental, cultural and economic.

1.2.3 Scope of Work

The geotechnical scope of work will vary for projects depending upon the size and complexity of the project. Development of a meaningful, project specific geotechnical scope of work is somewhat dependent upon the geotechnical information available at the time of scoping. During the Alternatives Analysis Phase, new alignments are being considered, and there most

likely will be no site specific subsurface information from earlier studies available at the time of scoping. However, a reasonable assessment of the geotechnical effort required at the Alternatives Analysis Phase can be done by the DGE using available information and a field view of the project area. Available information at this phase of the project will include some or all of the following:

- Conceptual roadway plans, profiles and cross-sections
- Subsurface information from a nearby project
- Topographic mapping
- Published soils and geologic information
- Aerial photography
- Soil and rock viewed in natural slopes/outcrops or nearby cuts
- Performance of existing cuts, fills and structures in the area

Guidance for using published information and performing site visits/reconnaissance is provided in [Chapter 2](#) of this publication.

A project located in mountainous terrain (i.e., most likely deep cuts and high fills) will require a fairly significant level of geotechnical effort during the Alternatives Analysis Phase to assess subsurface conditions and geotechnical considerations. Similarly, a fairly significant level of geotechnical effort can be expected during the Alternatives Analysis Phase if the project area is underlain by carbonate (limestone) bedrock, mined areas, acid bearing rock, complex/folded geology, if the project area contains colluvium/talus or landslide prone areas, or other complex or difficult geotechnical conditions exist.

Regardless of the size and complexity of the project, the following minimum geotechnical tasks are required and must be included in the scope of work for the Alternatives Analysis Phase of the project:

1. Review proposed plans, profiles and sections of alignment alternatives
2. Review available geotechnical information
3. Conduct field reconnaissance
4. Evaluate information/provide conceptual design recommendations
5. Compare (pros and cons) alternatives
6. Prepare Geotechnical Alternatives Analysis Report

Depending upon the specific circumstances and condition of the project, or anticipated complexity of subsurface conditions, additional geotechnical tasks may be necessary. Possible other geotechnical tasks that may need to be completed during the Alternatives Analysis Phase may include, but are not limited to:

7. Plan, implement, and supervise subsurface exploration program
8. Perform laboratory testing (soil, rock and water)
9. Interpret data from subsurface and laboratory testing programs
10. Perform engineering analyses

Performing these tasks during the Alternatives Analysis Phase is also beneficial because it will provide useful information for the geotechnical scoping process for the Preliminary Design Phase, and this information can most likely be used during the Preliminary Design Phase. Note that geotechnical tasks performed during the Alternatives Analysis Phase must be in accordance with other PennDOT manuals/publications, including Publication 10 (DM-1).

1.2.4 Required Geotechnical Tasks

As indicated above, several geotechnical tasks are required during the Alternatives Analysis Phase of a project. A detailed discussion of these tasks is provided below, and guidance for performing these tasks is included in subsequent chapters of this publication. These tasks are listed in the general order in which they should be performed, although some tasks can be performed simultaneously, or may be somewhat iterative.

While performing these tasks, it is important to communicate with other disciplines (i.e., roadway, environmental and bridge, etc.) involved with the project to ensure geotechnical issues related to their work are addressed and that the needs and constraints of these disciplines are considered. As examples, roadway designers will need conceptual cut and fill slopes for estimating limits of disturbance, ROW considerations, and soil/rock shrink/swell factors for earthwork balance. Bridge designers will need input on anticipated foundation types for structures.

Prior to starting the geotechnical work, it is desirable for a kickoff meeting to be held to:

- Review the geotechnical scope of work
- Estimate time frame to complete the work
- Discuss deliverables that will be prepared and the format of these deliverables
- Establish deliverable review process and anticipated progress/review meetings.

At a minimum, the PGM and the DGE should participate. For larger and more complex projects, it may be beneficial to have other individuals involved with the project participate. These may include consultant and PennDOT project managers, highway designers, bridge engineers, environmental engineers, and others. When all involved parties have a clear understanding of the anticipated, both general and specific, geotechnical activities from the onset, it will help to: ensure all necessary work is completed within the required timeframe, achieve the desired standard of quality, minimize the level of conflict between competing project needs.

1.2.4.1 Review of Plans, Profiles and Sections

The first geotechnical task is review of the conceptual plans, profiles and typical cross sections of the proposed alignments. These will provide an indication of the complexity of the project from a geotechnical standpoint. If conceptual plans indicate relatively gentle terrain and low proposed cut depths and fill heights, these alignments will generally be less complex geotechnically compared to alignments in steep terrain with proposed deep cuts and high fills. Landforms and terrain (i.e., geomorphology) are the first window or indicator of subsurface

conditions and should be studied carefully for the information it can offer. However, keep in mind that although steep terrain is an indication of a geotechnically complex alignment/project, geotechnical issues can also be present in areas of gentle terrain. These include karst topography (i.e., limestone geology), mining, soft soils and others. The geomorphology is only an indicator.

Review of the conceptual alignment plans will also provide an indication of areas where subsequent geotechnical tasks (i.e., reconnaissance and possibly subsurface exploration) should be concentrated. These areas include: deep cuts, high fills, low lying/wetland areas, areas of irregular topography, etc., that may provide an indication of other potential problem areas.

1.2.4.2 Review of Available Geotechnical Information

Once the proposed alignments are reviewed, available geotechnical data must be obtained and reviewed. Consultants must request from the Department, if available, geotechnical data obtained from previous studies, or geotechnical data from adjacent projects that may be useful. Additionally, research of published topographic, soil and geologic information must be done. At a minimum, the following information/sources must be reviewed:

- Topographic maps – both published and site specific project mapping, if available
- Aerial photography
- USDA Soil Survey
- Bedrock formations and structural features
- DCNR – Acid Bearing Rock map
- Open File Reports, if available/applicable
- Mine Maps, if available/applicable
- Other available information/sources

All pertinent geotechnical information obtained must be placed on project mapping to graphically show its location with respect to the proposed alignments. This provides a good summary of the information obtained, and is very useful for performing the field reconnaissance task. The PA Department of Conservation and Natural Resources, Bureau of Topographic and Geologic Survey, has an extensive library, and is an excellent source of geologic data. [Chapter 2](#) of this publication includes guidance on searching/obtaining available information.

1.2.4.3 Field Reconnaissance

After review of the proposed alignments and available geotechnical information, field reconnaissance of the proposed alignments, including structure locations, is required. Ideally, perform field reconnaissance after the alignments have been field located with flagging/stakes by others. Reconnaissance of all alignments is required, and the full length of the alignments must be viewed. However, concentrate on portions of the alignments identified during the review of available information (both geotechnical/geologic information and proposed construction) as potentially significant from a geotechnical point of view. As examples:

- Contours indicating contrasting steep and flat terrain or hummocky ground may indicate landsliding and/or rock outcrops

- Aerial photographs indicating lush vegetation may indicate seeps and springs
- Mapped sinkholes, landslides, mines, spoil piles, etc. must be viewed during reconnaissance.
- Areas of gently rolling terrain are often favorable, but they may also be an indicator of subsurface conditions unable to support steep slopes
- Rapidly changing contours and bodies of water (streams and rivers) may indicate rapidly changing subsurface conditions

If significant rock cuts are proposed along an alignment obtain representative rock discontinuity (bedding and joints) measurements from outcrops or cut slopes. These measurements can be used in a conceptual design stereonet analysis to estimate permissible rock cut slopes. Estimating slopes of deep rock cuts are valuable in determining approximate required limits of disturbance and earthwork balance.

Locate pertinent geotechnical features observed during field reconnaissance on project mapping. A camera and a hand-held GPS unit are very useful for performing this task, and a high level of accuracy is not necessary at this phase of the project. If necessary, flag areas of interest and use conventional survey to locate features on project mapping. **Reconnaissance performed during the Alternatives Analysis Phase is intended to identify significant features that may have an influence on the selection of the preferred alignment and assist in scoping for the next phase of design.** More detailed field reconnaissance will be conducted during the Preliminary Design Phase once the preferred alignment has been selected.

1.2.4.4 Evaluate Information/Provide Conceptual Design Recommendations

Once all available geotechnical information is obtained it must be evaluated in order to provide conceptual design recommendations. These conceptual design recommendations will be used by other designers (e.g., roadway, bridge, environmental, etc.) to help estimate the impacts and costs of the various alignments. Since site specific subsurface information may not be available, it is likely that the conceptual design recommendations will be based on engineering judgment and a careful assessment of available information and observations rather than direct engineering analyses. If sufficient information is available, analyses may be useful to develop conceptual design recommendations.

Communicate with other designers on the project when developing recommendations to determine their specific needs and constraints, and to communicate geotechnical related needs, constraints or limitations to them. Some conceptual design recommendations that will most likely be needed by others include:

- Embankment fill slopes and soil/rock cut slopes
- Shrink/swell factors for soil and rock
- Embankment foundation stabilization (overexcavation or ground improvement)
- Subgrade stabilization (overexcavation, lime, cement, etc.)
- Discussion of subgrade for pavement design
- Embankment surcharge, quarantine, wick/sand drains
- Void (e.g., limestone or mining) stabilization

- Non-standard treatments (e.g., extra depth sidehill benching, embankment toe trenches)
- Anticipated structure foundations
- Anticipated retaining wall types (MSE, cast-in-place, soldier beam and lagging, etc.)

It is important to consider any information or conditions that may provide an indication of potential problems. The information or conditions may have implications on alignment selection or future design and construction requirements. Scrutinize available information regarding the potential for encountering otherwise unforeseen conditions that may have major implications on the alignment selection, or future design and construction requirements.

1.2.4.5 Compare (Pros and Cons) Alignments

Finally, in addition to using gathered geotechnical information to provide conceptual design recommendations, use the collected information to assess the pros and cons of the various alignments from a geotechnical standpoint. This process may be one of the most valuable products of the geotechnical portion of the alternatives assessment, in that it provides decision makers with a simple and direct tool for comparing alternatives relative to geotechnical considerations. Favorable geotechnical conditions may include:

- Flat bedded geology
- Non-soluble bedrock
- Level or gently sloping terrain with low height fills and/or low depth cuts
- Medium to dense free draining granular overburden

Conversely, unfavorable geotechnical conditions may include:

- Steeply dipping and/or folded geology
- Soluble/carbonate/pinnacled bedrock (i.e., limestone, dolomite, marble)
- Irregular and/or steep terrain with high fills, sidehill fills and/or deep cuts
- Talus, loose granular overburden, and/or soft fine-grained overburden

1.2.4.6 Geotechnical Alternatives Analysis Report

Upon completion of the above tasks, and assuming no other geotechnical tasks are included in the scope of work, a Geotechnical Alternatives Analysis Report must be prepared. If other geotechnical tasks are required, as discussed in [Section 1.2.5](#) below, this report must include the results of those tasks as well.

The Geotechnical Alternatives Analysis Report must document all work conducted and information obtained during this phase. Present findings on project mapping when possible. Provide conceptual design roadway/bridge recommendations and construction considerations, and discuss the favorable and unfavorable geotechnical aspects associated with each alignment. Remember that it is important to present findings in a manner that serves the function and purpose of the study: assessing alternatives and the implications of the various alternatives from

a geotechnical impact perspective, but in context of the overall project goals and proposed construction.

A detailed discussion of the format and content of this report is provided in [Section 1.5](#).

1.2.5 Additional Geotechnical Tasks

Performing a subsurface exploration and laboratory testing program may be beneficial during the Alternatives Analysis Phase to better understand the subsurface conditions along the proposed alignments. Also, with site specific subsurface information, a higher level of engineering analyses can be performed to better estimate the geotechnical impacts on the project. Whether these tasks are undertaken requires careful consideration of the potential impact the geotechnical conditions may have on the project, and more importantly the impact that these conditions will have on selecting the desired alternative. If other factors (e.g., environmental, cultural, etc.) are driving the decision, subsurface explorations may be limited, or may concentrate on the more likely candidate(s) to be selected. The efforts should support the selection where possible, fully identify challenges with alternatives, and provide a basis of scoping for the next phase. In all cases, if/when sufficient geotechnical information has been obtained to eliminate an alternative (and other overriding considerations for that alternative do not exist) then geotechnical investigation for that alternative should cease immediately.

Performing these additional tasks (i.e., subsurface exploration, laboratory testing, and engineering analyses) during the Alternatives Analysis Phase should be considered for all projects, and are strongly encouraged for complex projects. These complex projects include those with alignments located in:

- Folded, pinnacled or carbonate geology
- Steep terrain
- Talus/colluvium slopes
- Landslide prone areas
- Mined (surface or deep) areas
- Acid bearing rock/soil, or other problematic deposits
- Urban areas

1.2.5.1 Subsurface Exploration Program

A subsurface exploration is done during the Alternatives Analysis Phase to better define the subsurface conditions along the proposed alignments. The subsurface exploration will normally include borings, but other investigation methods, like test pits or geophysics, should be considered when of significant value in assessing alternatives. These methods are discussed in detail in [Chapter 3](#) of this publication. Note that a “Subsurface Exploration Planning Submission” must be prepared, submitted and approved prior to performing this work.

Borings should be generously spaced (i.e., typically 1,000 feet or more) along the alignments because a detailed subsurface exploration for the selected alignment will be performed during the Preliminary and/or Final Design Phase. At least one boring should be

performed at each proposed structure location. Include a minimum two contingency borings in the core boring contract. More importantly, borings should be concentrated in the more complex areas of the alignments, or areas of proposed alignments that are of significant importance in alternative analyses, comparison and selection. Such areas may include items such as:

- Proposed deep cuts and/or high fills
- Steep terrain and landslide prone areas
- Deep and surface mined areas
- Areas of sinkholes and other karst features
- Presence of acid bearing rock
- Structure locations
- Waterways

The subsurface information obtained will provide soil and rock samples for visual classification/identification and laboratory testing, and will give an indication of groundwater level. The subsurface information will also aid to:

- Estimate permissible cut and fill slopes
- Determine if construction on steep terrain will be problematic and if special construction details will be likely
- Estimate soil shrink and rock swell factors
- Verify mine maps and determine condition of mined areas
- Determine condition of carbonate bedrock (i.e., solutioning, voids, pinnacles, etc.)
- Identify acid bearing rock
- Determine permissible and anticipated structure foundation types

Again, the subsurface exploration for an alignment shall be terminated if at any time sufficient information has been obtained to deem an alignment favorable or unfavorable. This determination will have to be made by the PennDOT Project Manager and the DGE in consultation with the PGM.

1.2.5.2 Laboratory Testing Program

A limited laboratory testing program will typically be sufficient for the Alternatives Analysis Phase because a detailed program for the selected alignment will be performed during the Preliminary and/or Final Design Phase. At minimum, natural water content, grain size distribution, hydrometer analysis and Atterberg limits of selected samples collected during the subsurface exploration must be performed in the laboratory to confirm field classifications. In some cases, consolidation, shear strength and corrosion testing may be necessary for the evaluation of alignment alternatives.

Rock and water testing may also be necessary during this phase. If acid bearing rock is anticipated based on published information or identified during drilling, testing should be done to estimate the acid producing potential. Rock strength and water corrosion testing may also be useful in some instances.

1.2.5.3 Interpretation of Data

Interpretation of the subsurface and laboratory testing information will be required to determine its significance on the proposed alignments. Certain qualitative judgments can be made directly from the test results. For example, loose, cohesionless soils should be cut fairly flat to be stable, are prone to erosion, and will experience immediate settlement if loaded. Cohesive soils may experience consolidation settlement if loaded, could be unstable if loaded too rapidly, and can be problematic to use as fill. In addition to qualitative judgment, engineering parameters can be estimated based on the data obtained from subsurface and laboratory testing programs. These parameters can be used in performing engineering analyses.

1.2.5.4 Engineering Analyses

Limited quantitative geotechnical analyses are typically required during the Alternatives Analysis Phase. However, in some instances, conceptual design analyses may be beneficial, particularly when site specific subsurface information is available. There are a variety of issues or conditions that may require or be highly beneficial to conduct preliminary analysis. For example, a slope stability analysis is used to determine if proposed fills and cuts (due to alignment changes) or existing slopes are stable, or if remediation is required to achieve a stable slope. Slope stability and stereonet analyses can also provide a better indication of required cut and fill slopes to help estimate limits of disturbance, compared to simply using engineering judgment, and these analyses can aid in identifying ROW needs and supporting preliminary cost estimates. Such cost estimates may be for comparing treatments for a given alternative, or comparing alternatives. In either case, preliminary analysis may be required to provide reliable estimates for cost. Additionally, consolidation settlement analyses can indicate if embankment quarantines, surcharges, and/or wick/sand drains may be required.

It is important that analyses performed are done only for conceptual design purposes to help identify favorable and unfavorable geotechnical aspects of the various alignments and to provide conceptual design recommendations needed by other designers. Detailed analyses will be performed on the selected alignment during the Preliminary and/or Final Design Phases.

As indicated in [Section 1.2.4.6](#), information obtained from subsurface exploration and laboratory testing programs, interpretation of this data, and results of engineering analyses must be presented in the Geotechnical Alternatives Analysis Report, and as with any engineering analyses, must support the conclusions and recommendation provided in the report.

1.2.5.5 Other Possible Tasks

Depending upon the proposed construction and site specific conditions, tasks in addition to those discussed above may be required. For example, an inventory and/or survey of water wells and springs to assess potential impacts from construction may be useful. If blasting is anticipated for construction, an evaluation of possible damage to nearby structures (houses, businesses, etc.) may be helpful for selecting the preferred alignment. These types of tasks should be identified during the scoping process, if possible, or otherwise supplemented to the scope of work when identified.

1.2.6 Structures

Much of the discussion in [Section 1.2](#) of this publication has focused on geotechnical engineering with respect to roadways. Structures, including structure foundations, are also an important consideration when studying the pros and cons of various roadway alignment alternatives or design options.

Also, refer to [Section 1.3.5](#) of this publication for discussion on the final design subsurface investigation for structure foundations.

1.2.7 Geotechnical Alternatives Analysis Phase Deliverables

The subsurface exploration plan submission, if appropriate, would include a summary and supporting documentation from the review of available geotechnical information, the results/findings of field reconnaissance, and the proposed subsurface exploration plan(s) for the alternatives (including proposed boring layout, proposed types and depth of borings, proposed depth into rocks) and any other pertinent information that lends justification and/or supports the proposed subsurface exploration. The submission must include a brief justification of the project scope and needs.

The Preliminary GER submission would include all subsurface exploration findings (boring logs, test pit logs, field testing results, etc.) and any available lab test results, analysis of subsurface findings, and lab test results relative to the proposed recommendations (including any preliminary recommendations for structures, as appropriate).

The Alternatives Analysis report would include the various foundation alternatives considered, their location, the advantages and disadvantages of alternative and approximate cost.

1.3 GEOTECHNICAL ENGINEERING - PRELIMINARY DESIGN PHASE

1.3.1 Introduction

The Preliminary Design Phase commences once the preferred alignment has been selected. The preferred alignment may have been selected from the Alternatives Analysis Phase of the project, or some variation thereof, in which case geotechnical engineering during the Preliminary Design Phase will be done as necessary to augment previously obtained information. If the project alignment was set from the onset (e.g., widening or rehabilitation projects) geotechnical engineering will commence during the Preliminary Design Phase. Similar to the Alternatives Analysis Phase, the Preliminary Design Phase is performed during Step 6 of the PennDOT Project Delivery Process.

The PGM and DGE shall communicate throughout the Preliminary Design Phase to ensure that the Department is aware of the subsurface conditions encountered, the results of the analyses performed, and the proposed recommendations. Considerable effort and valuable time can be saved by the PGM periodically updating the DGE instead of presenting all this

information to the DGE for the first time in the final report. It is desired that the process be highly collaborative, with the report reflecting such collaboration and interaction. When collaboration and communication are effective, the resulting report should require minimal review and editing.

1.3.2 Purpose

The purpose of geotechnical engineering during the Preliminary Design Phase/Preliminary Engineering is to:

- Permit development of preliminary recommendations (to be fully expanded in final design)
- Aid in the development of Line, Grade and Typical Sections, and the Design Field View Submission
- Support the environmental studies
- Develop a detailed Scope of Work for Final Design Phase (outcome of primarily the first bullet)

For major projects that involve construction of new alignment or total reconstruction of an existing facility, development of line, grade and typical section is generally the most extensive Preliminary Engineering activity and is a major part of the Design Field View Submission. Line, grade and typical section affect most aspects of a project's design and environmental issues because it establishes lateral and longitudinal impact limits. The project subsurface conditions, and ultimately the geotechnical design recommendations (i.e., permissible cut and fill slopes, shrink/swell factors, anticipated structure foundations, etc.), will influence line, grade and typical section, and sometimes significantly. Consequently, a good understanding of the subsurface conditions and some level of geotechnical engineering is critical during the Preliminary Design Phase of the project. Geotechnical effort during the Preliminary Design Phase also helps to define the environmental impacts of a project, and will allow the development of a detailed Scope of Work for Final Design. For small, less complex projects, only limited geotechnical engineering may be required during the Preliminary Design Phase; however, it is generally desirable to complete as much of the geotechnical work as possible during this Preliminary Design Phase. Project specific conditions and requirements will dictate the required level of effort.

1.3.3 Scope of Work

The geotechnical scope of work will vary for projects depending upon size and complexity of the project. Development of a meaningful, project specific geotechnical scope of work is dependent upon the geotechnical information available at the time of scoping. Projects with an Alternatives Analysis Phase will most likely have some level of site specific geotechnical information available, including a Geotechnical Alternatives Analysis Report, for developing a detailed scope of work for the Preliminary Design Phase. Projects that did not have any geotechnical work prior to the Preliminary Design Phase will have to rely on any existing sources of available information, along with a very clear understanding of the project objectives,

to prepare the scope of work. Refer to [Section 1.2.3](#) for guidance on possible sources of geotechnical information.

The amount of geotechnical work performed in the Preliminary Design Phase depends upon several factors, including: the nature and scope of the proposed construction, the contract agreement (scope and budget) between the Department and consultant, project schedule, certainty in the line, grade and location of structures, when approvals (e.g., Environmental Clearance, Design Field View, TS&L, etc.) are obtained, and others. **As much geotechnical engineering as possible is to be performed during the Preliminary Design Phase for several reasons, including:**

1. Adequate geotechnical engineering help ensure that the line and grade developed for the Design Field View Submission, and estimated environmental impacts, will best represent final conditions. This minimizes changes required to line and grade, typical sections, environmental impacts, etc. during the Final Design Phase.
2. Geotechnical recommendations are used by designers to complete their work. Early development of geotechnical recommendations allows designers to complete their tasks while understanding how geotechnical requirements impact other project requirements.
3. Meaningful geotechnical recommendations are dependent on adequate quantity and quality of information obtained from subsurface exploration and laboratory testing programs. These programs generally take considerable time to plan and execute (i.e., typically, a minimum six weeks for small projects and possibly six months or more for large projects). Performing a significant amount of work during the Preliminary Design Phase expedites completion and aids in improving the quality of the Final Design Phase.

The geotechnical tasks below must be included in the scope of work for the Preliminary Design Phase of the project. If an Alternatives Analysis Phase preceded the Preliminary Design Phase, several of these tasks should have already been performed, either partially or in entirety. If the tasks below were performed previously, do not duplicate work already performed. However, it is likely that some previous efforts may have to be augmented to provide the level of detail necessary for these tasks to advance the geotechnical design to the next phase.

1. Review proposed plans, profiles and sections
2. Review available geotechnical information
3. Conduct field reconnaissance
4. Plan, implement, and supervise subsurface exploration program
5. Plan and perform laboratory testing program (soil, rock and water)
6. Interpret data from subsurface exploration and laboratory testing programs
7. Perform engineering analyses
8. Provide geotechnical design recommendations
9. Prepare Preliminary Geotechnical Engineering Report

A detailed discussion of these geotechnical tasks is provided below. It should be noted that performing these tasks during the Preliminary Design Phase is also beneficial because it will provide useful information for the geotechnical scoping process for the Final Design Phase. Additionally, geotechnical information obtained during this phase will be used during the Final Design Phase, and gives a head start to a variety of other design activities that require geotechnical information, thereby assisting efficient project delivery.

1.3.4 Required Geotechnical Tasks

As indicated above, numerous geotechnical tasks are required during the Preliminary Design Phase of a project. Guidance for performing these tasks is included in subsequent chapters of this publication. These tasks are listed in the general order in which they should be performed, although some tasks can be performed simultaneously, or in some cases, may be iterative and incremental.

While performing these tasks, it is important to communicate with other disciplines (i.e., roadway, environmental, bridge, etc.) involved with the project to ensure geotechnical issues related to their work are addressed in an effective and timely manner. As examples, roadway designers may need special designs/details for steepened embankment and/or cut slopes to avoid wetlands or to stay within right of way. Bridge designers will need input on anticipated foundation types for structures and anticipated retaining wall types. It is absolutely crucial to avoid “the failure to communicate.”

Prior to starting the geotechnical work, a kickoff meeting shall be held to:

- Review the geotechnical scope of work
- Estimate time frame to complete the work
- Discuss deliverables that will be prepared
- Establish the deliverable review process and anticipated progress/review meetings.

Again, regular interaction and communication prior to preparation of deliverables (bullet #3), will greatly reduce efforts required for bullet #4. At a minimum, the PGM and the DGE shall communicate regularly. For larger and more complex projects, it may be beneficial to have other individuals involved with the project be included in these communications via meetings, conference calls, etc. These individuals may include consultant and Department project managers, highway designers, bridge engineers, environmental engineers, and others. All involved parties having a clear understanding of the anticipated geotechnical activities from the onset will help to ensure all necessary work is completed within the required timeframe.

If a consultant is performing this work, the product must be in accordance with the direction of, meet the needs of, and serve the client – the Department. Department staff should not forget that open communication, clear and frequent direction, reasonable expectations, and above all, consistency, are necessary for a consultant to provide an efficient and effective product. These are not mutually exclusive limitations or constraints; rather they form the basis for mutual respect and a successful team effort. Finally, both parties must understand that

“direction” does not equate to either lack of independent thought or action, nor decrees and demands. It is providing the necessary guidance leading to the ultimate acceptance of the product. The ultimate acceptance may sometimes require setting necessary limitations to facilitate acceptance and approval; however, such limitations must be accompanied by sound reasoning and justification.

1.3.4.1 Review of Proposed Alignment Plans, Profiles and Sections

The first geotechnical task is review of the proposed alignment plans, profiles and sections. As indicated in [Section 1.2.4.1](#) of the Alternatives Analysis Phase, this review will provide an indication of the complexity of the project and an indication of areas where subsequent geotechnical tasks (i.e., reconnaissance, subsurface exploration, etc.) should be concentrated.

1.3.4.2 Review of Available Geotechnical Information

Once the proposed alignment has been reviewed, available geotechnical data must be obtained and reviewed. If an Alternatives Analysis Phase was completed, this task should have already been completed and possibly augmented with a subsurface exploration during Alternatives Analysis. If so, review the Geotechnical Alternatives Analysis Report for pertinent information. If it appears all available geotechnical information for the proposed alignment was not previously reviewed, research, obtain and synthesize any necessary additional information.

If there was not an Alternatives Analysis Phase or this task was not completed during the Alternatives Analysis Phase, a thorough search and review must be completed. Consultants must request from the Department, if available, geotechnical data obtained from previous studies or projects, or geotechnical data from adjacent projects that may be useful. Additionally, research of published topographic, soil and geologic information must be done. At a minimum, the following information/sources must be reviewed:

- Topographic maps – both published and site specific project mapping, if available
- Aerial photography
- USDA Soil Survey
- Bedrock formations and structural features
- DCNR – Acid Bearing Rock map
- Open File Reports – if available/applicable
- Mine Maps – if available/applicable
- Other available information/sources

All pertinent geotechnical information obtained must be identified on the project map(s) to graphically show its location with respect to the proposed alignment. Regional mapping data such as geology, soils, mining, etc., can be shown on separate maps, provided the proposed roadway alignment is shown. Sources of discrete data should be identified by location on a map, preferably the project map(s), with the proposed alignment shown. Additionally, a discussion must be located in the proposed Subsurface Exploration Plan, along with any necessary summary of data, figures or appendices. This provides a clear summary of the information obtained. This

information is also very useful for performing the field reconnaissance task, and planning any necessary subsurface explorations. [Chapter 2](#) of this publication includes guidance on searching/obtaining available information.

1.3.4.3 Field Reconnaissance

Once available geotechnical information is reviewed, a thorough site reconnaissance must be completed. If an Alternatives Analyses Phase was completed, a site reconnaissance should have been performed. If this is the case, a more thorough reconnaissance of the proposed alignment is usually required during the Preliminary Design Phase depending upon the complexity of the project and the findings from previous reconnaissance. A thorough site reconnaissance is critical during the Preliminary Design Phase and prior to the preparation of the subsurface exploration program. It is worth noting that some level of site reconnaissance is necessary during each phase of design to be able to relate the site in context with the proposed design and construction. Foregoing this step in any phase is an invitation to missed opportunities and future problems.

If there was not an Alternatives Analysis Phase, or this task was not completed during the Alternatives Analysis Phase, a thorough field reconnaissance must be completed at this time. Ideally, perform field reconnaissance after the alignment has been marked with flagging/stakes by others. If staking is not complete, then adequate mapping must be available showing the proposed alignment over local topography, roadways, and other relevant and identifiable physical features and landforms. Reconnaissance of the full length of the alignment must be performed. Ensure that areas identified during the review of available information as potentially significant from a geotechnical point of view are closely viewed. As examples:

- Contours depicting steep terrain or hummocky ground may indicate landsliding and/or rock outcrops
- Lush vegetation seen on aerial photographs may indicate seeps and springs
- Mapped sinkholes, landslides, mines, spoil piles, etc. must be viewed during reconnaissance.

If rock cuts are proposed obtain rock discontinuity and structure measurements from outcrops or existing rock cut slopes. These are required for use in stereonet analyses to provide rock cut slope recommendations.

Locate pertinent geotechnical features observed during field reconnaissance on project mapping. A hand-held GPS unit is very useful for performing this task and may provide an acceptable level of accuracy. If necessary, flag areas of interest and use conventional survey to locate features on project mapping.

1.3.4.4 Subsurface Exploration Program

It is highly recommended and strongly encouraged to perform as comprehensive of a subsurface exploration program as possible during the Preliminary Design Phase. As previously indicated, meaningful geotechnical engineering, and corresponding recommendations, are highly

dependent on adequate and quality subsurface information, including sufficient laboratory testing of materials. Obtaining this information during the Preliminary Design Phase will help minimize design changes and environmental impacts related to geotechnical issues during the Final Design Phase, improve the overall quality of design, improve the flow and efficiency of the final design process, and result in fewer problems during construction. Additionally, the subsurface exploration program generally takes considerable time to plan and execute, and performing this work during the Preliminary Design Phase is helpful in expediting the overall Final Design Phase of the project.

The level of subsurface exploration performed will be dependent upon numerous things, including the certainty of the line, grade and/or structure locations. A comprehensive subsurface exploration program should be performed when these are fairly well established and it is unlikely that changes will be made. Conversely, if line and grade and/or structure locations are not well established and significant changes are possible, then a less comprehensive subsurface exploration program should be considered since some of the subsurface exploration effort may not be applicable to the revised/final line and grade. Requirements for subsurface explorations are included in [Chapter 3](#) of this publication.

Always perform a comprehensive subsurface exploration consistent with the project needs, project size, type of construction, anticipated geology and available subsurface information. Large, complex projects, such as new alignments or lengthy widening projects will generally require a more extensive subsurface exploration. A sufficient exploration must be performed to aid in the development of the line and grade, and estimate environmental impacts and right of way needs. Perform a subsurface exploration program that provides a reasonable understanding of the subsurface conditions. Investigations must be performed along the entire alignment. More detailed or closely spaced investigations must be performed in geotechnical areas of interest identified during the search of available information, reconnaissance, and previous investigations, and in areas of proposed high fills and deep cuts, or other potential problem, high risk, or high investment areas (e.g., structure foundations or areas of high cost construction where failures cannot be tolerated or would be costly and/or damaging). If a comprehensive subsurface exploration program is not performed during this phase, additional borings will be required during the Final Design Phase to finalize geotechnical design recommendations. Even if a comprehensive exploration is performed during the Preliminary Design Phase, some limited additional subsurface exploration may be required to finalize geotechnical recommendations and details during the Final Design Phase.

For small, less complex projects, such as short widening and rehabilitation projects, a single subsurface exploration program, performed either during the Preliminary or Final Design Phase, should be adequate for the project. If possible, it is preferred to perform the investigation during the Preliminary Design Phase. Utilize the Accelerated Acquisition of Drilling and Testing Service option presented in Publication 222, Section 4.5, if the estimated cost of the subsurface exploration program satisfies the requirements.

A comprehensive subsurface exploration for structures should also be performed as soon as possible. If structure layouts are approved (i.e., TS&L or Conceptual TS&L approval) it is recommended to perform a complete structure subsurface exploration during the Preliminary

Design Phase. At a minimum, perform at least one boring at each proposed bridge and culvert location during the Preliminary Design Phase. For multiple span bridges, it is recommended to perform two or more borings to obtain a subsurface profile along the bridge. Communicate with bridge engineers to estimate likely substructure locations. Consider performing boring(s) at each anticipated substructure location (minimum of two borings per substructure unit preferred); these borings would frequently be adequate for final foundation design. The number of borings required will vary depending on the size of the structure, the project scope, and the specific site specific geologic conditions. For retaining walls and sound barrier walls, at a minimum, space borings to obtain a general understanding of the subsurface conditions at the proposed walls. Structure borings will also likely be useful for geotechnical roadway design purposes.

The DGE must be kept informed of the progress and results of the subsurface exploration. In particular, any unusual or unforeseen conditions encountered must be immediately discussed with the DGE, especially findings that may require adjusting or modifying the approved subsurface exploration plan. Prior to the completion of the subsurface exploration, the findings must be discussed with the DGE to determine if additional explorations are necessary prior to demobilization. For large projects a meeting in the field to review soil and rock samples may be beneficial. Smaller projects may not require a meeting at the site. Electronic correspondence, an office meeting, and/or a telephone conference may be adequate.

The proposed subsurface exploration program must be reviewed and approved by the DGE. The subsurface exploration contract and work must be done in accordance with Publication 222. Specifications for proposed work that is not addressed in Publication 222 must be developed by the PGM and reviewed and approved by the DGE.

Environmental documents shall be reviewed, if available, and discussion with those performing the environmental work for the project is required to assess the potential for encountering hazardous waste on the project. If encountering hazardous waste during the subsurface exploration is anticipated, the subsurface exploration program must be performed under a Health and Safety Plan (HASP) as indicated in Publication 222. The HASP must be prepared by the PGM and reviewed and approved by the DGE. For complex projects, the HASP may also be reviewed by the District Environmental Manager. If hazardous waste is encountered or suspected during the subsurface exploration program, the exploration must be stopped immediately and the PGM must notify the DGE.

1.3.4.5 Laboratory Testing Program

As with the subsurface exploration, the laboratory testing program must be consistent with the project needs, size and type of construction, and above all, should reflect the findings of the borings. The amount of laboratory testing performed will generally be consistent with the level of the subsurface exploration program. If a large subsurface exploration program is performed, then a more extensive laboratory testing program is likely necessary. Conversely, if only a limited subsurface exploration is performed, then a limited laboratory testing program will most likely be sufficient. See [Chapter 4](#) of this publication for guidelines on laboratory testing.

All proposed laboratory testing must be reviewed and approved by the DGE prior to performing the tests. All testing must be conducted in a laboratory accredited by the AASHTO Materials Reference Laboratory (AMRL) for the individual test methods that are conducted. All original test data, calculations, graphical plots and other relevant information must be presented with the test results. The test method (AASHTO/ASTM) and sample source (SR, Section, boring number, sample number and depth) must also be indicated.

1.3.4.6 Interpretation of Data

All information obtained from subsurface exploration and laboratory testing programs performed must be interpreted for use in analyses. The biggest and most consistent omission (failure) in geotechnical designs is the poor use of available subsurface information and laboratory testing. Failure to fully and adequately use this information can and has resulted in poor designs, recommendations, details and specifications, and problems later during construction.

If a subsurface exploration and/or laboratory testing program were performed during this design phase, the information obtained from these must be interpreted for use in analyses. Interpretation includes identifying and delineating subsurface strata and assigning engineering properties and design parameters (i.e., internal friction angle, cohesion, unit weight, consolidation parameters, etc.). It is preferred and recommended that this interpretation be done during the Preliminary Design Phase, even if the analyses will not be completed until the Final Design Phase. Interpreting the subsurface information during the Preliminary Design Phase will provide the DGE adequate time to review the recommended subsurface engineering properties prior to performing analyses. Provide the interpretation (identification and delineation of subsurface strata) and recommended engineering properties and design parameters to the DGE for review and approval.

Subsurface information obtained from the investigation must be graphically shown on profiles and/or cross-sections where it aids in interpretation of subsurface conditions. Use gINT Software for creation of “soil fences” to be placed on the soil profile plans and cross sections. It may not be necessary to show all boring information on profiles and cross-sections. At a minimum, graphically display borings/information that are pertinent to performing analyses, developing details, etc. Where the terrain is flat a subsurface profile is usually adequate for presenting information. In steep or mountainous terrain presenting information on cross-sections is typically more useful and helpful to understand the subsurface conditions.

Use the results from the subsurface exploration and laboratory testing programs to assign engineering parameters to the various subsurface strata. Use laboratory and field test results (e.g., shear strength, consolidation, unconfined compressive strength (rock), etc.) to directly assign parameters instead of using published information. If these types of tests were not performed, use Standard Penetration Test (SPT) results, sieve analysis, Atterberg limits, hydrometer, etc. to estimate internal friction angle and unit weight from published information. Selection of the parameters must be consistent with the material encountered. If using SPT to estimate shear strength parameters, adequate laboratory classifications and hydrometer analyses must be performed to demonstrate consistency of selected shear strength parameters with

materials encountered. A more conservative approach must also be used where selecting shear strength parameters in this manner. If unconfined compressive strength tests are not conducted a similar approach in estimating compressive strength of rock must be used. Rock type, degree of weathering, quality and general physical condition must be considered, and a conservative approach should again be adopted. [Chapter 5](#) of this publication provides guidance on selection of soil and rock parameters.

1.3.4.7 Engineering Analyses

Similar to numerous other geotechnical tasks, the amount or level of geotechnical engineering analyses performed during the Preliminary Design Phase can vary. Complete geotechnical engineering analyses should be performed on those projects where line and grade are well established and a comprehensive subsurface exploration and laboratory testing program have been completed. As previously discussed, the advantages to this are there will likely be less changes to design and environmental impacts related to geotechnical issues during the Final Design Phase, and it will expedite the overall schedule of the Final Design Phase of the project. If complete analyses are not performed at this time, they must be completed during the Final Design Phase. Similarly, if comprehensive subsurface exploration and laboratory testing programs are not completed during the Preliminary Design Phase, detailed analyses must be done during the Final Design Phase. In general, as much of the geotechnical work should be completed during the Preliminary Design Phase as practical, to facilitate the smooth completion of final design for all other disciplines.

At a minimum, sufficient geotechnical engineering analyses must be performed to aid development of the Line and Grade and Typical Section Submission, and to support the environmental studies of the project. If complete geotechnical engineering analyses are performed at this time, provide all analyses necessary to justify geotechnical design recommendations and details. Prior to performing analyses, obtain approval from the DGE on the subsurface strata and engineering properties established from the interpretation of data (See [Section 1.3.4.6](#)).

Foundation design/analyses may be done during the Preliminary or Final Design Phase depending upon when structure layouts are approved, the contract agreement between the Department and the consultant, and when the subsurface exploration is performed.

1.3.4.8 Geotechnical Design Recommendations

The level of detail to which the geotechnical design recommendations will be developed will be dependent upon the level to which the subsurface exploration and engineering analyses have been done. If comprehensive investigations and analyses have been performed, it is recommended that the geotechnical recommendations be developed as fully as possible and provided at this time. If sufficient subsurface information and/or analyses are not available at this time, then only preliminary recommendations can be provided. At a minimum, preliminary recommendations needed to develop the Line and Grade and Typical Section Submission are required to be provided at this time, along with any other recommendations that may be needed by designers. When preliminary recommendations are provided, they should be conservative to

reduce the likelihood of changes during the final design phase from negatively impacting the project. For instance, use conservative cut and fill slopes to estimate right-of-way limits and environmental impacts. A complete list of tasks necessary to complete final design must be compiled to facilitate scoping of final design.

As previously discussed, communicate with designers using the geotechnical recommendations to understand how the recommendations are being applied, to make sure all necessary recommendations are provided, and to resolve conflicts early where recommendations are incompatible with other project needs. This may require revising geotechnical recommendations, or working with other disciplines to determine what changes should be made that best serve the project. Additionally, the PGM and DGE shall communicate throughout the development of geotechnical recommendations. Considerable effort and valuable time can be saved by the PGM and DGE periodically communicating (both directions) instead of presenting recommendations to the DGE for the first time in the final report.

1.3.4.9 Preliminary Geotechnical Engineering Report

The findings and recommendations from the above mentioned geotechnical tasks shall be summarized in a report. All supporting data, information, calculations, etc., must be included in the appendices of the report. Where appropriate use mapping, cross-sections, figures and tables to show pertinent and important information and recommendations. If a “Geotechnical Alternatives Analysis Report” was prepared for the project, include any pertinent information from this report in the “Preliminary Geotechnical Engineering Report”. Also in this report provide discussion of geotechnical work items (i.e., investigations, testing, analyses, etc.) that are recommended to be performed during the Final Design Phase.

Submit a draft of the report to the DGE for review and comment, and address all comments before submitting the final report for approval by the DGE. After all comments have been resolved and appropriate revisions confirmed, submit copies of the final approved report to the DGE in the agreed number and format.

1.3.5 Structures

Much of the discussion in [Section 1.3](#) of this publication has focused on geotechnical engineering with respect to roadways. Structures, including structure foundations, are also an important consideration during the Preliminary Design Phase. As discussed several times, it is highly recommended and preferred to perform geotechnical explorations as early in the design process as possible. Therefore, if preparation of the TS&L or Conceptual TS&L is part of the Preliminary Design Phase, then the subsurface exploration for structures should also be performed at this time.

1.3.6 Preliminary Design Phase Geotechnical Deliverables

The subsurface exploration plan submission would include a summary and supporting documentation from the review of available geotechnical information, the results/findings of field reconnaissance, the proposed subsurface exploration plan (including proposed boring

layout, proposed types and depth of borings, proposed depth into rocks, another pertinent information), and a preliminary lab testing plan., The submission must include a brief, but comprehensive, analysis/justification of the proposed boring plan and lab testing supported by available geotechnical information, field reconnaissance, and the project scope and needs.

The Preliminary GER submission would include all subsurface exploration findings (boring logs, test pit logs, field testing results, etc.) and lab test results, analysis of subsurface findings, and lab test results relative to the proposed recommendations (including any preliminary recommendations for structures, as appropriate).

For large projects, consideration should be given to requiring a “Geotechnical Findings Report”. This report would occur between the subsurface exploration plan submission and the preliminary GER Submission. The report would include all subsurface exploration findings (boring logs, etc.) and lab test results, and include a brief analysis of the results for discussion as to the proposed direction and elements for preliminary design. The information provided in the findings report (boring logs and lab test results) would not be repeated in the preliminary GER.

The above submissions should be adequate to cover the vast majority of projects. However, there may be other items/information and/or possibly separate submissions, if not covered above, if project specific needs dictate.

1.4 GEOTECHNICAL ENGINEERING – FINAL DESIGN

1.4.1 Introduction

Per DM-1, Design Field View Approval and Design Approval are given at the end of Step 6 of the PennDOT Project Delivery Process, and these are only given after Environmental Clearance. Once Design Approval is granted, Final Design, which is included in Step 7 of the Process, can begin. The amount of geotechnical engineering required during this step will be dependent upon the amount of work completed during the Preliminary Design Phase.

The PGM and DGE shall communicate with each other and other disciplines throughout the Final Design Phase to ensure that the Department is aware of the subsurface conditions encountered, the results of the analyses performed, the proposed recommendations fit project needs and constraints, and proposed recommendations are fully implemented in the design and contract documents. Considerable effort and valuable time can be saved by the PGM periodically updating the DGE instead of presenting all this information to the DGE for the first time in the final report.

1.4.2 Purpose

The two main purposes of geotechnical engineering during the Final Design Phase are to develop geotechnical recommendations for final design of the roadway and/or structures, and to prepare geotechnical contract documents (e.g., boring plan, profiles, details, special provisions, etc.). All remaining/outstanding geotechnical work (i.e., work not completed during the Preliminary Design Phase) must be completed during the Final Design Phase. In many cases,

considerable geotechnical work, including borings, laboratory testing and design calculations was completed during the Preliminary Design Phase; however, even in these cases, design calculations and finalizing plans, specifications and typical details are usually necessary during the Final Design Phase. Subsurface exploration and laboratory testing must also be completed at this time if it was not completed during the Preliminary Design Phase or if it is needed to augment exploration and testing that was already performed.

1.4.3 Scope of Work

The scope of work for the Final Design Phase shall be developed to complete the geotechnical engineering for the project. For large or complex projects, a considerable amount of geotechnical engineering was likely completed in the Alternatives Analysis and/or Preliminary Design Phase; any remaining or additional geotechnical work to be completed during the Final Design Phase should be well defined. On projects where some level of geotechnical engineering was completed, information should be available to prepare a scope of work. Projects that little or no geotechnical work performed prior to the Final Design Phase will rely on the available information discussed in [Section 1.2.3](#) for preparation of the scope of work.

As previously discussed, it is preferred and recommended to complete as much geotechnical work as possible during the Preliminary Design Phase. However, since there is some flexibility with when and to what level geotechnical tasks are completed, there will be variation in the Final Design scope of work from project to project. The below geotechnical tasks must be considered when preparing the scope of work for Final Design. Several of these tasks may have already been performed, either partially or completely, during the Preliminary Design Phase. Therefore, the scope of work must be tailored to reflect what work has already been completed versus what tasks (or portions of tasks) are remaining.

1. Review proposed plans, profiles and sections
2. Review available geotechnical information
3. Conduct field reconnaissance
4. Plan, implement, and supervise subsurface exploration program
5. Plan and perform laboratory testing program (soil, rock and water)
6. Interpret data from subsurface exploration and laboratory testing programs
7. Perform engineering analyses
8. Provide geotechnical design recommendations
9. Prepare geotechnical related contract documents (e.g., boring plan, profiles, details, special provisions, etc.)
10. Prepare Final Geotechnical Engineering Report

A detailed discussion of these geotechnical tasks is provided below.

It is important to again stress that any geotechnical document or report is to have a purpose in moving a project forward. Therefore, any report or deliverable should consist of two distinct components or phases: the part that describes what information was found, condition identified, results obtained or calculations performed, and the part done or proposed because of that information. In this manner, each deliverable serves a purpose and provides direct value in

moving the project ahead. No deliverable described in this document should deviate from, or not fulfill, this goal and purpose. As an example, item numbers 1 through 7 above identify some of the information and analyses obtained or performed, and items 8 and 9 identify what is to be done because of the information, findings and analyses, and the recommendations. Most importantly, the first set of items (findings and analyses) must support the recommendations.

1.4.4 Required Geotechnical Tasks

As indicated above numerous geotechnical tasks may be required during the Final Design Phase of a project. Tasks that were completed during the Preliminary Design Phase to a level that is sufficient for final design purposes do not have to be performed. Tasks that were performed during the Preliminary Design Phase, but not to a level sufficient to support final design recommendations, must be completed at this time. Requirements and/or guidance for performing these tasks are included in subsequent chapters of this publication. These tasks are listed in the general order in which they should be performed, although some tasks can be performed simultaneously.

While performing these tasks, it is important to communicate with other disciplines (i.e., roadway, environmental, bridge, etc.) involved with the project to ensure geotechnical issues related to their work are addressed. As examples, roadway designers may need special designs/details for steepened embankment and/or cut slopes to avoid wetlands or to stay within right of way. Bridge designers will need foundation types, recommendations and details for structures.

Prior to starting the geotechnical work, a kickoff meeting shall be held to:

- Review the geotechnical scope of work
- Estimate time frame to complete the work
- Discuss deliverables that will be prepared and the format of these deliverables
- Establish deliverable review process and anticipated progress/review meetings
- Discuss geotechnical contract documents to be prepared.

At a minimum, the PGM and the DGE shall attend. For larger and more complex projects, it may be beneficial to have other individuals involved with the project attend. These may include consultant and Department project managers, highway designers, bridge engineers, environmental engineers, and others. All involved parties having a clear understanding of the anticipated geotechnical activities from the onset will help to ensure all necessary work is completed within the required timeframe.

1.4.4.1 Review of Proposed Alignment Plans, Profiles and Sections

The first geotechnical task is review of the proposed alignment plans, profiles and sections. This review should have been completed during the Preliminary Design Phase, but must be done again to determine if changes have been made to the proposed line, grade and/or sections during or since preliminary design, to make sure there are no undetected or outstanding

issues that need to be addressed in the final phase, and to identify any potential issues that could arise as design progresses.

1.4.4.2 Review of Available Geotechnical Information

This task should have been completed during the Alternatives Analysis and/or the Preliminary Design Phases of the project. If already performed, review the Preliminary Geotechnical Engineering Report for pertinent information. If there appears that pertinent and beneficial geotechnical information for the proposed alignment may be missing or overlooked, research, obtain and review the necessary additional information. If for some reason this task was not completed during a previous design phase, a thorough search of available relevant and beneficial geotechnical information must be completed at this time in accordance with [Section 1.3.4.2](#). Such information may provide critical insight or evidence of problems or conditions that may have significant impact on the project or important design implications.

1.4.4.3 Field Reconnaissance

It is likely that field reconnaissance was performed during the Preliminary Design Phase of the project. If a thorough field reconnaissance was already performed, additional reconnaissance may not be needed. If only a cursory or no field reconnaissance was previously performed, a thorough reconnaissance must be completed at this time. A thorough site reconnaissance is critical and particularly necessary prior to the preparation of the subsurface exploration program. Refer to [Section 1.3.4.3](#) for field reconnaissance requirements. As with, and in conjunction with, available geotechnical information, observations may be made that can significantly impact project direction and decisions. A field view is also absolutely critical for designers to be able to make decisions and recommendations in full context and appreciation of the field conditions and constraints present.

1.4.4.4 Subsurface Exploration Program

As previously discussed, it is strongly recommended to perform as comprehensive of a subsurface exploration program as possible during the Preliminary Design Phase. Obtaining this information early in the project development will help minimize design changes and environmental impacts related to geotechnical issues during the Final Design Phase. Additionally, planning and performing a subsurface exploration program takes considerable time so completing this during the Preliminary Design Phase is helpful in expediting final design.

If it was not possible to perform a comprehensive subsurface exploration during the Preliminary Design Phase, or if additional subsurface information is needed, final explorations must be completed early in the final design phase. Adequate subsurface information must be obtained to develop an accurate stratigraphic model, estimate the engineering parameters of soil and rock, perform engineering analyses, and develop final design geotechnical recommendations. This applies equally to roadway and structures.

The DGE must be kept informed of the progress and results of the subsurface exploration. In particular, any unusual or unforeseen conditions encountered must be

immediately discussed with the DGE. Prior to the completion of the subsurface exploration, the findings must be discussed with the DGE to determine if additional investigations are necessary prior to demobilization. For large projects, a meeting in the field to review soil and rock samples may be beneficial. Smaller projects may handle this with electronic correspondence, an office meeting, and/or a telephone conference.

Requirements for subsurface explorations are included in [Chapter 3](#) of this publication. The proposed subsurface exploration program, presented in the “Subsurface Exploration Planning Submission”, must be reviewed and approved by the DGE. If a limited subsurface exploration is proposed to augment information obtained from a previous investigation, a less formal letter type submission may be submitted in lieu of the “Subsurface Exploration Planning Submission” outlined in [Section 1.5](#). The subsurface exploration contract and work must be done in accordance with Publication 222. Specifications for proposed work that is not addressed in Publication 222 must be developed by the PGM and reviewed and approved by the DGE. Utilize the Accelerated Acquisition of Drilling and Testing Services option presented in Publication 222 (Section 4.5) if the estimated cost of the subsurface exploration program satisfies the requirements.

Environmental documents shall be reviewed, if available, and discussion with those performing the environmental work for the project is required to assess the potential for encountering hazardous waste on the project. If encountering hazardous waste during the subsurface exploration is anticipated, the subsurface exploration program must be performed under a Health and Safety Plan (HASP) as indicated in Publication 222. The HASP must be prepared by the PGM and reviewed and approved by the DGE. If hazardous waste is encountered or suspected during the subsurface exploration program, the exploration must be stopped immediately and the PGM must notify the DGE.

1.4.4.5 Laboratory Testing Program

If a comprehensive subsurface exploration program was performed during the Preliminary Design Phase, then a comprehensive laboratory testing program was also likely performed at this time. If not the case, then the final laboratory testing program must be performed during the Final Design Phase. All soil and rock testing needed for selecting geotechnical parameters to perform geotechnical analyses must be completed during this design phase. See [Chapter 4](#) of this publication for laboratory testing program requirements and guidelines.

All proposed laboratory testing must be reviewed and approved by the DGE prior to performing the tests. All testing must be conducted in a laboratory accredited by the AASHTO Materials Reference Laboratory (AMRL) for the individual test methods that are conducted. All original test data, calculations, graphical plots and other relevant information must be presented with the test results. The test method (AASHTO/ASTM) and sample source (SR, Section, boring number, sample number and depth) must also be indicated.

1.4.4.6 Interpretation of Data

All information obtained from subsurface exploration and laboratory testing programs performed must be interpreted for use in analyses. The biggest and most consistent omission (failure) in geotechnical designs is the poor (or complete lack of) use of available subsurface information and laboratory testing. Failure to fully and adequately use this information can and has resulted in poor designs, recommendations, details and specifications, and problems later during construction. Refer to [Section 1.3.4.6](#) of this publication for more discussion/requirements.

1.4.4.7 Engineering Analyses

Complete geotechnical engineering analyses for roadway and structures must be performed during the Final Design Phase if they were not completed during the Preliminary Design Phase. Necessary analyses must be completed to develop and support final design geotechnical recommendations. Geotechnical analyses that may be needed include: embankment settlement, embankment and cut slope stability, stereonet/rock cut slope stability, rockfall catchment, foundation recommendations, and more.

Engineering analyses performed during the Preliminary Design Phase must be reviewed to assure that they are consistent with final design requirements and revised as needed. Changes to line and grade, the addition of subsurface information or other factors may require revisions to previously performed analyses.

1.4.4.8 Geotechnical Design Recommendations

Final design geotechnical recommendations must be provided at this time. Recommendations provided during the Preliminary Design Phase must be reviewed and revised or updated as necessary. Recommendations may include: shrink/swell factors for soil and rock; estimation of suitability of soil for embankment use; permissible cut and fill slopes; details for sidehill benching, embankment toe trenches and undercuts, subgrade transitions and/or special treatments, and slope treatments; subgrade strength (CBR) and treatment; and foundation recommendations.

As previously discussed, communicate with designers that are using the geotechnical recommendations to understand how the recommendations are being applied, to ensure that they are consistent with project needs and constraints, to ensure that they do not conflict with other project needs or features, and to make sure all necessary recommendations are provided. Additionally, the PGM and DGE shall communicate throughout the development of geotechnical recommendations to facilitate smooth project development and to prevent delays and avoidable changes and modifications. Considerable effort and valuable time can be saved by regular communication between the PGM and the DGE instead of presenting recommendations to the DGE for the first time in the final report.

1.4.4.9 Final Geotechnical Engineering Report

The findings and recommendations from these geotechnical tasks shall be summarized in a report. All supporting data, information, calculations, etc., must be included in the appendices of the report. Where appropriate use mapping, cross-sections, figures and tables to show pertinent and important information and recommendations.

If a “Preliminary Geotechnical Engineering Report” was prepared for the project, include any pertinent information from this report in the “Final Geotechnical Engineering Report”. If the majority of the geotechnical work was performed during the Preliminary Design Phase and is documented in the “Preliminary Geotechnical Engineering Report”, a “Final Design Geotechnical Engineering Report” may not be needed if no or minimal information was obtained or revisions to recommendations were made during the Final Design Phase. Alternatively, an addendum to the “Preliminary Geotechnical Engineering Report” may be adequate. The addendum must contain the information obtained, analyses performed, and recommendations developed during the Final Design Phase. The type of report or addendum to be prepared should be discussed during development of the scope of work for the Final Design Phase.

Submit a draft of the report to the DGE for review and comment, and address all comments before submitting the final report for approval by the DGE. The record copy of the final report is to be submitted to the DGE in paper and approved electronic format. After all comments have been resolved and appropriate revisions confirmed, submit an electronic copy (CD or DVD in .pdf format) of the final report to the Chief Geotechnical Engineer. Individual Structure Foundation Geotechnical Reports for each structure must also be prepared and submitted during the Final Design Phase if not previously done.

Requirements and guidance for preparing reports are included in [Section 1.5](#) of this publication.

1.4.4.10 Geotechnical Contract Documents

Geotechnical related contract documents, including: subsurface exploration plans, subsurface profiles, subsurface cross-sections, details and special provisions, must be developed during the Final Design Phase. These documents must be provided in the Final Geotechnical Engineering Report for review by the DGE. Address all comments prior to incorporating into the contract documents. Also, provide structure related contract documents, including structure boring tracings, special provisions and details, in the Structure Foundation Report.

The PGM and DGE shall review, individually or jointly, the contract documents (e.g., plans, cross-sections, typical details and specifications) to ensure that the geotechnical recommendations have been properly incorporated, and that no geotechnical issues were overlooked or misunderstood during design development. Cross sections must be reviewed to verify that cut and fill slopes are acceptable, and that features (e.g., stormwater ponds, drainage swales, etc.) above and below slopes do not adversely impact performance of the slope.

1.4.5 Structures

Final structure foundation design recommendations must be provided during either the Preliminary or Final Design Phase of the project. As previously discussed, numerous factors will determine during which phase these recommendations are provided, but, when possible, structure foundation recommendations should be provided during the Preliminary Design Phase. [Section 1.3.5](#) of this publication discusses foundation recommendations for structures. Reference requirements in DM-4, Section 1.9.4 for streamlined and standard foundation submission requirements as well as foundation plan requirements.

1.5 GEOTECHNICAL ENGINEERING SUBMISSIONS

1.5.1 Introduction

There are six (6) types of geotechnical submissions that are prepared for Department projects. The reports required for a specific project will depend upon the type, size and complexity of the project, and whether structures are involved. The six types are:

1. [Subsurface Exploration Planning Submission](#)
2. [Geotechnical Alternatives Analysis Report](#)
3. [Preliminary Geotechnical Engineering Report](#)
4. [Final Geotechnical Engineering Report](#)
5. [Subsurface Findings and Parameters Report - Optional](#)
6. [Structure Foundation Report](#)
7. [Foundation Design Guidance Report](#)

Normally, at least two different types of reports will be required for a project, and for projects which include the study of alternate roadway alignments, as many as five different types will be required. This section of the publication discusses the purpose of the reports, when they are required, and the content and format of the reports.

1.5.2 Purpose

Geotechnical submissions serve various needs. One is to have a record of the geotechnical information obtained. Some projects, and in particular, complex ones: can take years to progress from the start of design to construction; may be unexpectedly stopped during the design process and later resumed; are completed by a variety of individuals (PennDOT and consultants); and/or are performed under different contract agreements. Therefore, in order to have a clear and concise record of information gathered for use during the design and construction phases of the project, geotechnical reports are required.

Geotechnical reports are also required to provide information and recommendations in order to move the design process forward. For example, as discussed below, the purpose of the “Subsurface Exploration Planning Submission” is to provide justification and obtain Department approval to proceed with a subsurface exploration program. Another reason geotechnical reports are required is to provide geotechnical recommendations for the design of roadways and

structures. The reports present data, analyses and the judgment used to justify the recommendations. When consultants prepare the reports, they are used by the Department to review and approve the recommendations. Once approved, the reports are used by highway, bridge, environmental and other engineers to design the facilities. Lastly, the geotechnical reports can be useful during construction and maintenance phases. If unforeseen conditions are encountered, or if some aspect of the project requires a modification/redesign, the geotechnical reports may provide useful information.

1.5.3 Subsurface Exploration Planning Submission

A Subsurface Exploration Planning Submission (SEPS) must be prepared and submitted when a subsurface exploration program (borings, test pits, geophysics, etc.) is necessary for roadway and/or structure design. This report is required to be submitted during each design phase (i.e., Alternative Analysis, Preliminary Design and Final Design Phases) in which a subsurface exploration program is proposed. The SEPS may be submitted after the structure footprint and core boring locations are determined during the streamlined TS&L process. If a project requires the preparation of this document in two or more design phases, subsequent reports should be prepared by using pertinent information from the previously prepared report(s), as well as pertinent information obtained during the previous subsurface exploration program(s).

When a subsurface investigation is the result or consequence of a previous phase of work (e.g. Preliminary Design) the SEPS may be incorporated into the previous design phase deliverable (i.e., may be included as part of the Preliminary Design GER) since the proposed investigation would, in effect, be a recommendation from the previous phase of work. This will eliminate unnecessary separate submissions and maintain the continuity and flow of work.

Structure borings for foundation design require a separate, individual SEPS for each structure unless the DGE recommends combining submissions due to close proximity of structures with similar non-complex geotechnical conditions. In accordance with DM-4, Section 1.9.3, this plan must be submitted with the TS&L Report for bridges and culverts. A SEPS is also required for retaining walls and noise walls, but the report for these types of structures is submitted prior to the TS&L submission. The submission requirements discussed in this section include the requirements set forth in DM-4, Section 1.9.3.

A single submission should be prepared when a limited number of roadway borings associated with a structure replacement/widening are proposed or when a limited number of structure borings are proposed (e.g., during the Alternatives Analysis and/or Preliminary Design Phase). When a limited number of borings are proposed to supplement a previous subsurface exploration program, a letter or memorandum may be sufficient to present and justify the additional subsurface exploration program. The geotechnical scope of work for the project should clearly indicate the requirements of the SEPS(s).

Department approval of the SEPS is required prior to performing the developing the core boring contract.

1.5.3.1 Submission Format and Content

The required format and content of the SEPS are shown below. Note that this submission may be in the form of a report or a less formal transmittal letter, as dictated by the scope and needs of the project. When submitted as a transmittal letter, all the information indicated below must be included/discussed.

Cover Page

Table of Contents

[1.0 Introduction](#)

[2.0 Proposed Construction](#)

[3.0 Information Search and Findings](#)

[4.0 Reconnaissance](#)

[5.0 Proposed Subsurface Exploration](#)

[6.0 Anticipated Laboratory Testing Program](#)

[7.0 References](#)

[Figures](#)

[Appendices](#)

Cover Page – The cover page must contain the following information:

- Title: Subsurface Exploration Planning Submission
- State Route and Section Number
- ECMS No.
- Structure (if applicable)
- Name of County, Pennsylvania
- Township
- Prepared for: Pennsylvania Department of Transportation
Engineering District _-0
- Prepared by: Design Consultant Name
- Month and year report submitted

Table of Contents – All pages in the report must be numbered.

1.0 Introduction – Provide a brief description of the project, including:

- Location of the project and reference to Site Location Map (In remote areas show the location of the nearest city or major intersection to aid in locating the project).
- Scope of work/tasks performed

In addition to a brief verbal description of project location, provide a Site Location Map either embedded in the text of the report or as a separate figure. Provide a single map with adequate detail for someone to drive to the project. Use Google maps or a topographic map annotated with SR numbers and roadway names.

2.0 Proposed Construction – Provide a brief description of the proposed project construction, concentrating on the features related to geotechnical investigation/design.

3.0 Information Search and Findings – This section shall contain information obtained from published sources regarding the topographic and subsurface conditions at the site. Details on performing this task are included in [Chapter 2](#) of this publication. Material reviewed and discussed in this section must include:

- Project Features Map (topographic map) showing contours and surface features (streams, rivers, strip mines, etc.). This map must be scaled to concentrate on the project area.
- Physiographic Setting
- In very limited situations, discuss United States Department of Agriculture (USDA) soil types. Since soils described on this map are surficial (i.e., only up to 5 feet in depth) and pertain to agriculture, they typically will not be relevant to engineering applications. If the top 5' of materials are used to support the roadway or are used as fill in other areas of the project, an abbreviated discussion may be warranted. Do not provide a detailed description of each soil type; only discuss those soil types and properties that are significant to the project geotechnically. If site specific soil information from previous investigation(s) is available USDA soil information should not be discussed.
- Geology – Provide a geologic map delineating bedrock formations and structural features (e.g., faults, anticlines, synclines, etc.). In the text discuss rock type(s) and geologic structure (i.e., bedding and jointing), if available. If applicable, and the information is available provide a stratigraphic column. Engineering geology - Provide general information, including: ease of excavation, drilling rate, and cut slope and foundation stability.
- Well data (where applicable)
- Review of aerial photography – include several different years, if available. Discuss pertinent findings, changes over time and conditions of potential impact and/or concern.
- Pertinent geotechnical findings from published reports and maps (e.g., mining, sinkholes, landslides, acid bearing rock, etc.).
- Pertinent information from review of existing roadway and structure plans and boring logs for subsurface information, geotechnical treatments, structure foundations, etc. Also, review available structure inspection reports, scour reports, etc., and discuss pertinent findings relative to potential project impacts and the exploration plan.
- Environmental issues – Include information provided/obtained by others during environmental evaluations of the project. Discuss any impacts or findings relevant to the investigation plan.

- Economic considerations – Provide discussion on mineable coal seam(s), high quality aggregate, or other materials that may be encountered on site that have economic value and may require economic compensation.
- Consultation with local residents, contractors, and Department personnel that have knowledge of the area. Discuss any relevant findings that may impact or influence subsurface exploration or proposed design and construction.
- Consultation with District Geotechnical Units for review of existing geotechnical information.
- Utility Considerations – provide distances to, or locations of, adjacent overhead utilities on Boring Location Maps.

4.0 Reconnaissance - This section shall document the findings from reconnaissance performed at the site. Details on performing this task are included in [Chapter 2](#) of this publication. Observations in this section must include information concerning the following items that may impact the exploration or are pertinent to the proposed design and construction.

- Topography, vegetation, seepage, etc.
- Soil types exposed in slopes, stream/river banks and beds, etc.
- Rock types exposed in outcrops, stream/river banks and beds, etc.
- Performance of existing facilities (roadways, structures, etc.), at or nearby the project site
- Indications of slope movements, mine subsidence, sinkholes/karst features
- Access for drilling equipment and type of required equipment anticipated (e.g., truck, track, all terrain, skid, barge, low-head rig, etc.)
- Types and location of overhead and underground utilities

5.0 Proposed Subsurface Exploration Program – This section of the report provides recommendations for the proposed subsurface exploration program. The proposed exploration program should reflect findings from the information search and field reconnaissance in context of the needs for the proposed design and construction activities. Requirements for subsurface explorations are included throughout this chapter of the publication, and additional detailed discussions on subsurface explorations are included in [Chapter 3](#) of this publication. Information in this section of the report must include:

- Type of investigation(s) proposed – borings, test pits, geophysics, etc.
- For structures, discussion of anticipated foundation type(s) and justification for number and depth of borings
- For roadways, provide table (if needed) indicating proposed cut depth/fill height and justification for number, depth and types of borings and sampling
- Access for drilling equipment and type (e.g., truck, track, all-terrain vehicle, skid, barge, etc.)
- Utility conflicts or concerns with conducting borings

- Maintenance and protection of traffic requirements
- Recommendation for mandatory pre-bid site meeting
- Special procedures required for drilling in or near streams/rivers (i.e., restrictions on time of year due to protected aquatic life, filtering drill wash water, Aids to Navigation (ATON))
- Need for Health and Safety Plan (HASP)
- Need for permits

6.0 Anticipated Laboratory Testing Program – This section of the report provides recommendations for the anticipated laboratory testing program. The final laboratory testing performed will be dependent upon the findings from the exploration (soil and rock types and conditions encountered), and must be approved by the DGE. For example, consolidation settlement testing will typically only be performed if soft, cohesive soil is encountered and collected in an undisturbed sample (e.g., Shelby tube). In this section of the report, provide an estimate of the number and types of tests proposed. Additionally, discuss the reason(s) for performing the tests. A detailed discussion of laboratory testing is provided in [Chapter 4](#) of this publication.

7.0 References – It is preferred that references used to prepare report/recommendations be included directly within the report text. A formal listing is not necessary. If references cannot be included in the text provide a list of the references at the end of the report. Any information from these references relevant to the justification or recommendations of the exploration plan that cannot be readily incorporated into the body of the report must be included in the figures section or an appendix as appropriate.

Figures – Embed figures directly in the report when possible and appropriate. Place figures that cannot be easily embedded within the report text in this section. Only include figures that are “value added”, and include as much information as possible on the same figure to limit the number of figures. The following figures are required in this report:

1. Site Location Map – As discussed in the Introduction section of this report, this figure must be a large enough scale so that someone not familiar with the site can locate it. Only one site location map that provides adequate information to locate the site should be provided. Annotate the map as necessary to aid in site location.
2. Project Features Map – As discussed in the Information Search section of this report, this figure must be scaled to focus on the immediate project area. This figure, possibly a topographic map, must include contours and surface features (streams, rivers, roads, strip mines, etc.).
3. Geologic Map – Annotate with location of project.

Other “value added” figures should also be included in this section of the report. These figures may include:

- USDA Soils Map – This map typically will not be useful and should not be included. Include only if site specific soils information (i.e., obtained from previous investigation) is not available and if USDA soil information is pertinent to geotechnical aspects of the project.
- Pertinent maps from Open File Reports (i.e., sinkhole maps), Mine Maps, Landslide Maps, Acid Bearing Rock Map, etc.
- Plans for existing roadway/structures showing subsurface information, geotechnical details/treatments, foundation types, etc.

Appendices – Two appendices (A and B) are required for this report. A third appendix (C) is required if preparation of a boring contract is included in the scope of work. Additional appendices may be included to present pertinent data or information. This data or information may include: well data, existing plans, boring logs, a General Plan and Elevation drawing from the preliminary TS&L in the report, etc.

Appendix A – This appendix presents the details of the proposed subsurface exploration. This appendix must include:

1. Boring Location Plan - This figure is required to present the specific locations of subsurface explorations (borings, test pits, cone penetrometer tests (CPT), geophysical survey(s), etc.). Scaling must not be smaller than 1" = 50 ft., but should be expanded to fill an entire sheet with all pertinent information. Use multiple sheets if necessary. This figure must be created from project mapping that shows the proposed roadway(s) and structure(s). If available, contours, utilities and topographic features shall be shown on the plan to help determine if there are utility conflicts, MPT requirements, access issues, etc. If contours are not available from project mapping, prepare a duplicate boring location plan on a topographic map scaled so the location and approximate elevation of borings is easily identified. Scaling should not be smaller than 1" = 50 ft. Use multiple sheets if necessary.
2. Attachment 1 – Schedule of Proposed Borings from Publication 222.
3. Engineer's Drilling Cost Estimate – Prepare a cost estimate of the proposed subsurface exploration program. Use pay items included in the PennDOT Standard Subsurface Boring, Sampling and Testing Contract, which is also included in Publication 222, as well as pay items associated with any required special provisions.

Appendix B – This appendix must include photographs of the site taken during the field reconnaissance. For roadway projects, include photos of pertinent geotechnical features and observations, including performance of existing roadways, outcrops, landslides, sinkholes, etc. For structures include photos of proposed structure site, and photos of existing structures for replacement/widening projects.

Appendix C – If preparation of a Test Boring Contract is part of the scope of work, a draft version of the contract must be included in this appendix. Also, include in this appendix the Letter of Interest proposed to be sent to PennDOT Prequalified Geotechnical Drilling Contractors, and the Intent to Enter letter(s) sent to property owners.

A transmittal letter must accompany the report. The transmittal letter from the design consultant to the Department shall note the contractual requirements with respect to the report and the status of the report (i.e., is it a draft or final report, etc.). The report shall be presented on standard letter sized paper. Plans, profiles, cross-sections and figures that are not legible on standard letter sized paper must be presented on 11x17 foldout sheets. Bullet points, tables, figures, sections, etc. shall be used where possible to clearly and concisely present the findings and recommendations.

1.5.4 Geotechnical Alternatives Analysis Report

A Geotechnical Alternatives Analysis Report is required when alternative alignments are being studied and geotechnical input is needed to help select the preferred alignment. This report documents the geotechnical information gathered during this phase of the project. The report also provides conceptual geotechnical design recommendations for use by other designers (roadway, environmental, bridge, etc.) to help estimate the impacts of the geotechnical conditions on the various alignments. Additionally, favorable and unfavorable geotechnical features associated with the various alignments are presented in the Geotechnical Alternatives Analysis Report.

1.5.4.1 Submission Format and Content

The required format and content of the Geotechnical Alternatives Analysis Report are as follows:

- Cover Page
- Table of Contents
- [1.0](#) Introduction
- [2.0](#) Summary of Recommendations and Alignment Considerations
- [3.0](#) Geotechnical Analyses and Interpretation of Data
- [4.0](#) Environmental Concerns
- [5.0](#) Economic Considerations
- [6.0](#) Site Investigation
 - 6.1 Available Information
 - 6.2 Aerial Photography
 - 6.3 Reconnaissance
- [7.0](#) Subsurface Exploration
 - 7.1 Description
 - 7.2 Roadway Subsurface Conditions

7.3 Structure Subsurface Conditions

7.4 Laboratory Testing Program

[8.0 References](#)

[Figures](#)

[Tables](#)

[Appendices](#)

Cover Page – The cover page must contain the following information:

- Title: Geotechnical Alternatives Analysis Report
- State Route and Section Number
- ECMS No.
- Name of County, Pennsylvania
- Township Name
- Prepared for: Pennsylvania Department of Transportation
Engineering District _-0
- Prepared by: Design Consultant Name
- Month and year report submitted

Table of Contents – All pages in the report must be numbered.

1.0 Introduction – Provide a brief description of the project, including:

- location of the project and reference to Site Location Map
- project description/proposed construction
- scope of work/tasks performed

In addition to a brief verbal description of project location, provide a Site Location Map either embedded in the text of the report or as a separate figure. Provide a single map with adequate detail for someone to drive to the project. Use Google maps or a topographic map annotated with SR numbers and roadway names. If the map scale permits, the figure must indicate the locations of the alignments being considered.

2.0 Summary of Recommendations and Alignment Considerations – This section of the report provides “conceptual” geotechnical design recommendations and presents geotechnical considerations that may influence the alignment selection process.

The geotechnical recommendations provided in this section are conceptual at this phase of the project since the overall project design is at a conceptual level, limited subsurface information will be available, and detailed analyses are typically not performed. However, these recommendations are important because they will be used by other designers (roadway, bridge, environmental, etc.) on the project to better determine impacts and costs of the various alignments. During development of the recommendations it is important to communicate with other

designers to understand what recommendations are needed and their significance to the alignment selection process. Recommendations must include, as applicable:

- Suitability of onsite materials (soil and rock) for use in embankments
- Need for special materials, like rock for embankments, and if material is available on site
- Shrink and swell factors for earthwork balance estimate
- Permissible embankment slopes
- Need for special embankment treatments that will add cost. For example: extra depth sidehill benching, toe trenches, undercuts, etc.
- Need for settlement treatment (wick/sand drains, quarantine or surcharge)
- Pavement subgrade parameter(s) for conceptual pavement design
- Pavement subgrade stabilization (overexcavation, lime, cement, etc.)
- Permissible cut (soil and rock) slopes
- Rockfall protection (i.e., catchment zone, fence, etc.)
- Anticipated structure foundation types
- Any anticipated problems or required special treatments for structure or embankment foundations
- Handling/treatment of acid bearing rock
- Mine void stabilization
- Treatment of sinkholes and karst topography
- Other geotechnical issues/features that will impact alignments or costs

3.0 Geotechnical Analyses and Interpretation of Data – This section of the report provides justification for the recommendations given in the previous section of the report. At this phase of the project minimal analyses may have been performed so justification for some of the recommendations may be based on published information, experience from previous projects, or commonly used/accepted practices. Also, discuss in this section the relative costs of various geotechnical alternatives relative to the entire project. For example, a specific geotechnical treatment may be expected to cost more than other alternative treatments, but the more costly treatment results in a reduction in the total cost of the roadway alignment alternative.

4.0 Environmental Concerns – This section of the report provides information on environmental findings from other studies that may have an impact on the geotechnical aspects of the project. For example, impact of landfills/dump sites, contaminated areas, wetlands, mine spoil, etc. within proposed embankment or cut areas.

5.0 Economic Considerations – This section of the report provides information regarding the presence of “valuable” materials that are located within the project limits that may impact project costs. For example, the presence of mineable coal

seams, high grade limestone, or other materials of value, may have a cost impact on an alignment under consideration.

6.0 Site Investigation – This section of the report summarizes the geotechnical information obtained during the review of available/published references and from reconnaissance. If a “Subsurface Exploration Planning Submission” was prepared, pertinent information from Section 3.0 (Information Search) and Section 4.0 (Reconnaissance) of this report should be used in addition to any information obtained after preparation of the “Subsurface Exploration and Planning Submission”.

6.1 Available Information - This section shall contain information obtained from published sources regarding the topographic and subsurface conditions at the site. Details on performing this task are included in [Chapter 2](#) of this publication. Information in this section must include:

- Project Features Map (topographic map) showing contours and surface features (streams, rivers, strip mines, etc.). This map must be scaled to concentrate on the project area.
- Physiographic Setting – Not required if presented in “Subsurface Exploration Planning Submission”.
- United States Department of Agriculture (USDA) soil types. Provide a map delineating the soil types. A detailed description of each soil type is not necessary; only discuss in detail those soil types and properties that are significant to the project geotechnically. If site specific soil information from borings is available USDA soil information is not required.
- Geology – Provide a geologic map delineating bedrock formations and structural features (e.g., faults, anticlines, synclines, etc.). If project site is located within one formation and no structural features are present, a map is not required. Provide general bedrock descriptions, including: type, color, bedding, weathering, and jointing. If applicable provide a stratigraphic column, for example when coal seams are expected to be encountered.
- Engineering geology - Provide general information, including: ease of excavation, drilling rate, and cut slope and foundation stability.
- Well data
- Pertinent geotechnical findings from published reports and maps (e.g., mining, sinkholes, landslides, acid bearing rock, etc.).
- Consultation with local residents, contractors, and Department personnel that have knowledge of the area.

6.2 Aerial Photography – This section shall present the findings from the review of aerial photography. Several different years, if available, shall be reviewed.

6.3 Reconnaissance - This section shall document the findings from reconnaissance performed at the site. Include pertinent photographs taken during reconnaissance, either embedded in the report or included in an appendix. Details on performing this task are included in [Chapter 2](#) of this publication. Observations in this section must include:

- Topography, vegetation, seepage, etc.
- Soil types exposed in slopes, stream/river banks and beds, etc.
- Rock types exposed in outcrops, stream/river banks and beds, etc.
- Performance of existing facilities (roadways, structures, etc.) at or nearby the project
- Indications of slope movements, mine subsidence, sinkholes/karst features
- Any other features or observations that may impact or have relevance to an alternative

7.0 Subsurface Exploration – Indicate “Not Applicable” if a subsurface exploration program was not performed during this phase of the project.

7.1 Description – Provide a brief description of the subsurface exploration, including:

- Purpose for performing the exploration(s)
- Type of exploration(s) performed (i.e., borings, test pits, geophysics, etc.)
- Number of borings and test pits, coverage area of geophysics, etc.
- Contractor that performed the work
- Approximate date(s) exploration was conducted
- Consultant that observed/inspected the work

7.2 Roadway Subsurface Conditions – Provide a summary of the subsurface conditions along the roadway alignments. A detailed discussion is not needed. Use tables and subsurface profiles/sections to help summarize and present the information.

7.3 Structure Subsurface Conditions - Provide a summary of the subsurface conditions at structure locations. A detailed discussion is not needed. Use tables and subsurface profiles/sections to help summarize and present the information.

7.4 Laboratory Testing Program - Provide a summary of the laboratory test results. A detailed discussion is not needed. Detailed laboratory test reports will be provided in an Appendix. Use tables if needed to help summarize and present the information.

8.0 References – It is preferred that references used to prepare report/recommendations be included directly within the report text. A formal listing is not necessary. If

references cannot be included in the text provide a list of the references in this section of the report. Any information from these references relevant to the justification or recommendations of the exploration plan that cannot be readily incorporated into the body of the report must be included in the figures section or an appendix as appropriate.

Figures – Embed figures directly in the report when possible and appropriate. Place figures that cannot be easily embedded within the report text in this section. Only include figures that are “value added”, and include as much information as possible on the same figure. The following figures are required in this report:

1. Site Location Map – As discussed in the Introduction section of this report, this figure must be a large enough scale so that someone not familiar with the site can locate it.
2. Project Features Map – As discussed in the Information Search section of this report, this figure must be scaled to focus on the immediate project area. This figure must include contours and surface features (streams, rivers, roads, strip mines, etc.).
3. Geologic Map
4. Subsurface Exploration Plan (if applicable) – This figure must show locations of borings, test pits, geophysics, etc.

Other “value added” figures should also be included in this section of the report. These figures may include:

- USDA Soils Map – This map typically will not be useful and generally should not be included. Include only if site specific soils information (i.e., obtained from previous investigation) is not available and if USDA soil information is pertinent to geotechnical aspects of the project.
- Maps from Open File Reports (i.e., sinkhole maps), Mine Maps, Landslide Maps, Acid Bearing Rock Map, etc.

Tables – Include tables that are not or cannot be embedded in the text of the report. This includes summary tables of material that may also be presented in the appendices. For example, a summary table for laboratory test results would be presented in this section while the individual test results and work sheets would be presented in the appendices.

Appendices – Present pertinent data or information in the appendices. This data/information may include:

- Boring logs or other subsurface exploration reports/records
- Laboratory test results
- Subsurface profiles and cross-sections – required if borings were performed
- Existing Plans
- Well Data

- Calculations/Analyses
- Details
- Photographs
- Supporting reference materials

A transmittal letter must accompany the report. The transmittal letter from the design consultant to the Department shall note the contractual requirements with respect to the report and the status of the report (i.e., is it a draft or final report, etc.). The report shall be presented on standard letter sized paper. Plans, profiles, cross-sections and figures that are not legible on standard letter sized paper must be presented on 11x17 foldout sheets. Bullet points, tables, figures, sections, etc. shall be used where possible to clearly and concisely present the findings and recommendations.

1.5.5 Preliminary and Final Geotechnical Engineering Reports

1.5.5.1 Introduction

The main purposes of the Preliminary and Final Geotechnical Engineering Reports are to document the geotechnical work performed during the Preliminary and Final Design Phases and to present design recommendations for use by other designers (e.g., roadway, bridge, environmental, etc.) on the project. The main focus of these reports is the roadway aspects of the project, although subsurface information at structure locations, conceptual foundation alternatives, and/or a summary of foundation recommendations/type are also included in these reports.

As indicated several times throughout this publication, the geotechnical work performed during the Preliminary and Final Design Phases of projects will vary depending upon several factors. Consequently, the contents of the Preliminary and Final Geotechnical Reports will vary, and in some cases only one of these reports may be required. The decision as to whether both a preliminary and final design report is required will depend on the scope of the project. The report(s) required to be prepared for a project should be determined during the scoping phase.

Since the geotechnical tasks performed during the preliminary and final design phases of a project are often similar, particularly for large/complex projects, the geotechnical report format for both preliminary and final design is the same. When geotechnical reports are prepared for both preliminary and final design, the final design report shall include all pertinent information from the preliminary report so that the final design report documents all geotechnical data, analyses, recommendations, etc. performed for the project. It is not intended that the preliminary report be repeated in its entirety. Instead, any subsurface data (including boring logs), lab test data, pertinent analyses and calculations, and any other relevant findings must be included and properly located in the final design report. Some of this information may reside in the appendices while other information may have to be incorporated into the appropriate sections of the text of the report.

1.5.5.2 Influence of Project Size and Complexity

For large, complex projects, both reports will most likely be required. For these types of projects the Preliminary Geotechnical Engineering Report will mainly provide geotechnical recommendations to:

- Help set the line and grade
- Identify impacts (right-of-way, environmental issues, drainage)
- Develop preliminary and/or conceptual recommendations
- Develop the final design phase geotechnical scope of work.

Subsequently, the Final Geotechnical Engineering Report will provide geotechnical recommendations for final design of the roadway and will present geotechnical related contract documents (i.e., subsurface boring plan, subsurface profiles and cross sections, detail, special provisions, etc.).

For smaller, less complex projects, frequently only one report, either the Preliminary or the Final will be required. This depends on the amount and/or complexity of the geotechnical engineering work necessary. For instance, if the majority of the geotechnical work is performed during the Preliminary Design Phase, then a Preliminary Geotechnical Engineering Report must be prepared. If minimal additional, follow-up geotechnical work is needed during the Final Design Phase, this work can be documented in an addendum to the Preliminary Geotechnical Engineering Report instead of preparing a complete Final Geotechnical Engineering Report.

Geotechnical exploration results and recommendations for structure projects are presented in the Structure Foundation Report. When structure projects include “minor” roadway improvements/modifications, exploration results and recommendations for the roadway should also be included in the Structure Foundation Report, if possible. Alternatively, an abbreviated Preliminary or Final Geotechnical Engineering Report may be acceptable, if approved by the DGE.

Again, it is absolutely critical that the scope of work be clearly defined, and it is crucial that the DGE be involved in this process.

1.5.5.3 Submission Format and Content

The Preliminary and Final Geotechnical Engineering Reports must be prepared using the following format:

- Cover Page
- Table of Contents
- [1.0](#) Introduction
- [2.0](#) Summary of Geotechnical Design Recommendations
- [3.0](#) Geotechnical Analyses and Interpretation of Data
- [4.0](#) Soil, Rock and Geologic Setting
- [5.0](#) Reconnaissance

6.0 Subsurface Exploration

6.1 Description

6.2 Roadway Subsurface Conditions

6.3 Structure Subsurface Conditions

6.4 Laboratory Testing Program

7.0 Environmental Concerns

8.0 Economic Considerations

9.0 References

Figures

Tables

Appendices

- Subsurface Exploration Plan, Profiles and Cross-Sections (Prefer Subsurface exploration plans, profiles and cross sections to be submitted in 11 x 17 format separately from the text of the GER and in designated/specified electronic drafting format.)
- Typed Engineers Boring Logs Calculations
- Core Box Photo Log
- Driller's Logs
- Full Laboratory Test Reports
- Detailed Analysis of Data (i.e., parameter selection)
- Calculation Briefs
- Details
- Special Provisions
- Other

1.5.5.4 Report Overview

Each of these items/sections will be covered in more detail below, however, the following brief summary is presented below to provide an overview of what is expected for the report and indicate the purpose of the various report sections, how they complement each other, and how all sections/contents are to ultimately support Section 2.0 – Summary of Geotechnical Design Recommendations.

The “Introduction” provides a brief overview of the project and contents/purpose of the report. The overview should be brief, yet sufficient for any user of the report to gain a general understanding of the proposed construction and how the focus of this report fits with the overall project requirements. The “Summary of Geotechnical Design Recommendations” section provides a straight-forward, but comprehensive listing of the proposed geotechnical recommendations. The recommendations must be clear and specific with adequate detail to allow recommendations to be readily incorporated into the overall design by others (e.g., highway, bridge and environmental engineers, CADD operators, etc.). The details must thoroughly cover “what is to be done” and “where it is to be done”, and references to required special provisions and details, where applicable, must be provided.

Section 3.0, “Geotechnical Analysis and Interpretation of Data” provides the link between Section 2.0 and the remainder of the report. This section provides a comprehensive

summary of the findings from all sources of exploration and analyses of the data, including selected geotechnical parameters and references to (and locations of) pertinent supporting data (e.g. acquired subsurface information, laboratory test results, calculations and analyses). Recommendations provided in Section 2.0 should be thoroughly explained/justified in Section 3.0, so that the reviewer(s) can understand why the recommendations are proposed. The comprehensive supporting subsurface information (e.g., boring logs and laboratory test results) is provided in the “Subsurface Findings” section and the appendices. Sections 4.0 and 5.0 provide a summary of available published geologic information and site observations from reconnaissance, identifying any pertinent site conditions that may impact geotechnical design and/or construction.

Section 7.0 provides information as required to address any environmental concerns and/or requirements that must be accommodated in design or construction. Section 8.0 is primarily to address mineral right issues, where mineable materials are present, or other economic issues that could impact design recommendations.

The figures, tables and appendices provide supplemental information critical to the design, but that must either be presented in a graphical format (figures or tables), that does not lend itself to incorporation into the body of the report, consists of supporting information or analysis for one of the other report sections, cannot efficiently be presented in one of the main report sections due to size, or consists of a work product that is referenced in the recommendations that must be incorporated in the contract or plans. These last three sources make up the appendices which include subsurface exploration plan, profiles and cross sections, typed engineer’s boring logs, full lab test results, detailed analysis of data (i.e., parameter selection), calculation briefs, details, special provisions, or other pertinent information that supports the findings, analyses or recommendations.

Cover Page – The cover page must contain the following information:

- Title: FOR DESIGN PURPOSES ONLY
Preliminary OR Final Geotechnical Engineering Report
- State Route Number and Section Number
- ECMS Number
- Name of County, Pennsylvania
- Prepared for: Pennsylvania Department of Transportation
Engineering District _-0
- Prepared by: Design Consultant Name
- Month and year report submitted

Table of Contents – All pages in the report, including Appendices, must be numbered.

1.0 Introduction – The introduction shall include:

- Purpose of the report
- Geotechnical scope of work
- Location of the project

- Description of the project.

In addition to a brief verbal description of the location of the project, provide a map either embedded in the report or as a separate figure. The figure shall cover an area large enough for someone not familiar with the project to be able to locate it. The project description shall focus on those aspects of the project associated with the geotechnical work.

2.0 Summary of Geotechnical Design Recommendations – This section provides a summary of all geotechnical recommendations.

- Recommendations shall be specific and concise. A detailed discussion of the recommendations and supporting analyses/data will be provided in Section 3.0.
- Sections 2 and 3 must be coordinated so that rationale behind or justification of recommendations provided in Section 2.0 can be easily found in Section 3.0.
- Present recommendations in an orderly manner. Group recommendations by subject (e.g., embankment foundations, embankment slopes, soil cut slopes, rock cut slopes, pavement subgrade, etc.) and use list format whenever practical.
- When applicable, provide Station to Station and offset limits for the recommendations. Recommendations must be meaningful and understandable to other disciplines (e.g., highway designers, CAD technicians, etc.) so they can be properly applied and presented in the contract documents.
- Use tables to efficiently present recommendations where appropriate and whenever they may aid in assuring accurate follow through into the contract documents.
- When applicable provide a typical detail or figure to help explain recommendations.
- If current Department construction specifications (Publication 408) are adequate, only reference the specification section number. Do not recite text from the specification.
- Reference Special Provision(s) associated with recommendations.

Provide the following recommendations where applicable. Additional recommendations may be needed depending upon the specifics of the project.

2.1 Embankment Construction (Fills)

- (a) Suitability of on-site soil and rock obtained from projects cuts for use in embankments with respect to Publication 408, Section 206.
- (b) Location and depth of unsuitable soil with respect to Publication 408, Section 206.
- (c) Location and depth of topsoil available on site.

- (d) Shrink and swell factors for soil and rock.
- (e) Recommended fill slope angles (i.e., 2H:1V, 1.5H:1V, etc.).
- (f) Need for “select” embankment materials (e.g., rock lining on embankment slopes, zoned earth and rock embankments, rock embankment, etc.) and if material is available on site or if borrow is required.
- (g) Special requirements for fill placed below proposed structures (e.g., no rock within pile or drilled shaft window, etc.)
- (h) Benching details for placing embankment on sloping ground or widening existing embankments.
- (i) Embankment foundation preparation requirements (e.g., overexcavation and replacement, proof rolling, geotextile/geogrid, ground improvement, etc.).
- (j) Treatment of seeps/springs beneath proposed embankments.
- (k) Treatment of existing building/structure foundations, slabs, etc. prior to embankment placement.
- (l) Treatment of existing pavement beneath proposed embankment.
- (m) Need for embankment toe trenches/keys.
- (n) Subsidence/sinkhole treatment (mining related and karst).
- (o) Settlement quarantine locations, estimated time periods and/or surcharge embankment requirements, wick drains, etc.
- (p) Instrumentation and monitoring requirements (piezometers, settlement plates, inclinometers, etc.).
- (q) Surface and subsurface drainage requirement.
- (r) Erosion protection during and after construction.

2.2 Cut Slope Construction (Soil and Rock)

- (a) Recommended cut slope angles.
- (b) Rockfall catchment (ditch, fence, netting, etc.) recommendations.
- (c) Treatment of material in cut face for erosion, durability, differential weathering, etc.
- (d) Treatment of seeps in soil cut slopes.
- (e) Treatment for concentrated areas of runoff down soil cut slope face.
- (f) Location and quantity of unsuitable rock for embankment construction.
- (g) Location, depth and treatment of hazardous material.
- (h) Location of potential acid bearing rock (ABR) and treatment/handling recommendations.
- (i) Soil/Rock subgrade transition (longitudinal and transverse) details.
- (j) Presence of boulders that may affect excavation.
- (k) Special blasting requirements, such as presplitting or vibration monitoring.
- (l) Instrumentation and monitoring requirements (e.g., slope inclinometers, etc.).
- (m) Surface and subsurface drainage requirements

- (n) Erosion protection during and after construction

2.3 Subgrade Recommendations

- (a) Recommended subgrade parameters for pavement design.
- (b) Subgrade treatment/stabilization (e.g., overexcavation and replacement, geotextile/geogrid, lime, etc.)
- (c) Pavement drain recommendations
- (d) Need for subgrade-subbase separation
- (e) Special consideration (for example, at-grade construction on alluvial/colluvial soils with high water table, construction over mined areas)

Note that pavement design is not part of the GER and must be documented in a separate report.

2.4 Structure Subsurface and Foundation Summary – Detailed discussions of subsurface conditions and foundation recommendations at structure locations are presented in the Structure Foundation Geotechnical Report. Provide only a summary of structure information in this section, including:

- (a) Subsurface conditions at structure locations
- (b) Anticipated structure foundation types
- (c) Approach embankment and spread footing settlement potential, and need for settlement quarantine period, surcharge, wick drains, etc.
- (d) Global stability concerns

2.5 Miscellaneous

- (a) Treatment of abandoned wells and springs.
- (b) Anticipated construction issues (not related to specific recommendation listed above) and recommended remedy.
- (c) Subsidence related recommendations (e.g., exploratory air-track drilling, impervious lining of swales in karst topography, void filling, etc.).
- (d) Stormwater pond recommendations (e.g., location with respect to stability of proposed cuts and embankments, requirements of fill for embankment construction)
- (e) Special requirements for culvert/drainage pipe (e.g., need for foundation preparation, placing pipe on steep grades, etc.).
- (f) Consideration of waste areas with respect to stability and settlement of embankments, cuts and structures.
- (g) Sinkhole Considerations and sinkhole treatments

2.6 Additional exploration/study (for Preliminary Design GER only)

- (a) Recommend need for additional subsurface explorations (i.e., borings, CPT, geophysics, etc.) and/or laboratory testing.
- (b) Discuss geotechnical items that are considered critical to roadway design of the project and require additional study/analyses.

These types of recommendations shall only be included in the Preliminary Design GER and not the Final Design GER. Needed additional investigations or studies identified during the final design phase must be immediately discussed with the DGE.

3.0 Geotechnical Analyses and Interpretation of Data – This section contains detailed discussion of the recommendations provided in Section 2.0 of the GER. The discussion shall include data and parameters used, assumptions made, analyses performed, alternatives considered, etc. to fully explain/justify the recommendations. Organize/arrange this section in the same manner as Section 2.0 so that justification for recommendations can be easily found.

4.0 Soil, Rock and Geologic Setting – This section shall contain information obtained from published sources regarding the subsurface conditions at the site. This information is most likely available from the Subsurface Exploration Planning Submission. Information provided in this section shall include:

- Physiographic Setting
- United States Department of Agriculture (USDA) soil types. A map delineating the soil types shall be included. A detailed description of each soil type is not necessary; only discuss in detail those soil types that are significant to the project geotechnically.
- Bedrock stratigraphy. Provide general descriptions, including: type, color, bedding, weathering, and jointing.
- Structural geology features (e.g., faults, anticlines, synclines, etc.).
- Engineering geology. Provide general information, including: ease of excavation, drilling rate, cut slope stability, and foundation stability.
- Pertinent geotechnical findings from published reports and maps (e.g., landslides, sinkholes, mining, etc.).
- Discussion of subsurface conditions encountered relative to/versus anticipated geology.

5.0 Reconnaissance – This section shall document the findings from reconnaissance performed at the site during this phase and earlier design phases. Include pertinent findings from reconnaissance that help support geotechnical design recommendations. Provide photos where appropriate and helpful to explain conditions observed during reconnaissance.

6.0 Subsurface Findings – This section shall be divided into two subsections, including Subsurface Conditions and Laboratory Testing Summary.

6.1 Subsurface Conditions – This section shall document all subsurface explorations (i.e., borings, geophysics, CPT, test pits, etc.) performed at the site, both during Preliminary and Final Design. Include in this section discussion of instrumentation (i.e., piezometers, inclinometers, etc.) and field tests (i.e., vane shear, pressure meter, etc.) performed. A plan view showing the locations of all borings, CPT's, test pits, geophysical surveys, instruments, etc. shall be included in the Appendix of the report. Logs and field instrumentation/test reports from the investigations must also be included in an Appendix.

The text of this section must include a discussion of the types of Investigations performed and instrumentation installed. Also, a brief discussion of the subsurface conditions encountered shall be included. Do not include a detailed written discussion of the subsurface conditions since this is typically difficult to comprehend. Use subsurface profiles and cross-sections, and tables as needed, to present detailed subsurface information.

6.2 Laboratory Testing Summary – This section shall document all laboratory testing performed for the project. Provide a brief discussion of the tests performed and the results. Utilize tables as needed to summarize laboratory test results. All lab test reports must be included in an Appendix.

7.0 Environmental Concerns – This section shall document any environmental concerns associated with the project that are related to geotechnical activities (e.g., earthwork, foundations, etc.). Typically, this section will summarize findings from reports prepared by others on the project, in particular, environmental scientists/engineers. Any geotechnical recommendations associated with environmental concerns must be provided in Section 2.0. Environmental items that shall be considered include:

- Wetlands
- Acid Bearing Rock (ABR)
- Hazardous Waste
- Underground Storage Tanks
- Contaminated/Unclean Fill

8.0 Economic Considerations – This section shall discuss any economic aspects of the geotechnical work associated with the project. Items that shall be considered are:

- Presence of mineable coal or other minerals in proposed cuts or beneath the proposed roadway.
- Presence of aggregate grade rock in proposed cuts or beneath proposed roadway.

- Mass balance of the project earthwork. Is waste material marketable for use as a construction material elsewhere? For borrow projects, is it possible and more economical to adjust grade or modify/flatten cut slopes to obtain necessary material from within project area.
- Possible cost saving options to limit right-of-way acquisition (e.g., use of reinforced soil slopes to steepen embankment slopes, steepened cut slopes, retaining walls, profile adjustment, etc.).

9.0 References – It is preferred that references used to prepare the report/recommendations be included directly within the report text. A formal listing is not necessary. If references cannot be included in the text provide a list of the references in this section of the report. Any information from these references relevant to the justification or recommendations provided in the report that cannot be readily incorporated into the body of the report must be included in the figures section or an appendix as appropriate.

Figures - Embed figures directly in the report when possible and appropriate. Place figures that cannot be easily embedded within the report text in this section. Only include figures that are value added, and include as much information as possible on the same figure. The following figures are required in this report and most likely can be obtained from a previously prepared report (e.g., Subsurface Exploration Planning Submission or Geotechnical Alternatives Analysis Report), although some modifications/additions may be required:

1. Site Location Map – As discussed in the Introduction section of this report, this figure must be a large enough scale so that someone not familiar with the site can locate it.
2. Project Features Map – This figure must be scaled to focus on the immediate project area. This figure must include topographic contours and surface features (streams, rivers, roads, strip mines, etc.).
3. Geologic Map

Other “value added” figures shall also be included in this section of the report. These figures may include: Maps from Open File Reports (i.e., sinkhole maps), Mine Maps, Landslide Maps, Acid Bearing Rock Map, etc.

Tables – Include tables that are not or cannot be embedded in the text of the report. This includes summary tables of material that may also be presented in the appendices. For example, a summary table for laboratory test results would be presented in this section while the individual test results and work sheets would be presented in the appendices.

Appendices – Appendices shall include data, analyses, special provisions, details, and other necessary supporting information referenced in the report. The following appendices are required:

Appendix A – Subsurface Exploration Plan, Profile and Cross-Sections

The Subsurface Exploration Plan must show the locations of all borings, test pits, CPT's, instrumentation, etc. Also, provide any subsurface profiles and cross-sections prepared in this appendix.

The information obtained from the subsurface exploration and laboratory testing programs must be included in the contract documents for the contractor's use for bidding/constructing the project. Publication 14M (DM-3), Chapter 5 provides requirements for the Soil Profile Plans.

Graphically present the subsurface information on the roadway profile. Borings shall be shown at the station and elevation drilled along the profile, with the boring designation, station, offset, and ground surface elevation indicated above the boring. The graphical display shall include standard symbols to identify soil and rock type, and AASHTO classifications and rock type shall be indicated adjacent to the boring. The graphical display of the boring shall also include SPT N-values, rock core recovery, Rock Quality Designation (RQD), and groundwater readings. Laboratory test results shall be shown in a table on the same sheet the boring is displayed. A lab test number shall be assigned to each test, and that number shall be referenced on the graphical display adjacent to the corresponding stratum. Strata in a boring that are not laboratory tested must be referenced to a lab test number that is similar. Prefer subsurface exploration plans, profiles and cross sections to be submitted in 11x17 format separate from the text portion of the report.

Appendix B - Typed boring logs, test pit logs, CPT records, etc.

Appendix C – Full Laboratory Test Reports

Provide individual laboratory test reports for each test performed. A Professional Engineer licensed in the Commonwealth of Pennsylvania must certify that all testing was conducted in a laboratory accredited by the AASHTO Materials Reference Laboratory (AMRL) for the individual test methods conducted.

Appendix D - Calculations, Calculation Briefs, and Detailed Analyses

All calculations/analyses used to develop the recommendations presented in the report must be included. Calculations/analyses must meet the following requirements:

- Where appropriate, a calculation brief should be included when it is necessary to describe the rational or specific approach used in the design or to clarify the calculation procedure.

- When appropriate, necessary or required, detailed analyses should be included that cannot be efficiently included in Section 3.0 – Analyses and Interpretation of Data.
- Include a list of method(s) and reference(s). Copies of charts or tables used in the analysis are to be included with the calculations. Tables or charts contained in the DM-4 or AASHTO Specifications need not be duplicated.
- Be neatly prepared and organized
- Show all work, equations, description of variables, assumptions and units
- Be checked and signed by an independent reviewer
- Be presented in a verifiable format
- Calculations done with a spreadsheet must be accompanied by a representative hand calculation that comprehensively verifies results of the spreadsheet. The hand calculation must meet all the above bullet requirements.
- When calculations are performed using a computer program, input parameters and output must be clearly presented.
- Except when using a Department indicated or Department required program (e.g., SLIDE v7.0), supporting program documentation and representative sample hand calculation(s) must be provided. This information must: describe the program; indicate the program logic, design assumptions and limitations; and show that hand calculations match program results.

Justification must be provided for all soil and rock strata parameters that are used in the calculations. For soil, these may include drained and undrained shear strength (friction angle and cohesion), unit weight (moist and saturated), Young's modulus, consolidation properties (C_c , C_r , C_v) and others as needed. For rock, these parameters may include unconfined compressive strength, unit weight and others as needed. This justification must be provided in the beginning of the Appendix. It is preferred that justification of parameters be based on direct measurement from laboratory and/or field tests. If correlations are used, provide a calculation that meets all the above requirements. Subsequent calculations that use these parameters shall reference this soil parameter justification/calculation.

Appendix E - Typical Details

Appendix F - Special Provisions

Others as needed

A transmittal letter must accompany the report. The transmittal letter from the design consultant to the Department shall note the contractual requirements with respect to the report and the status of the report (i.e., is it a draft or final report, are any addendums anticipated, etc.). The Preliminary and Final Geotechnical Engineering Reports shall be presented on standard

letter sized paper. Plans, profiles, cross-sections and figures that are not legible on standard letter sized paper must be presented on 11x17 foldout sheets. Bullet points, tables, figures, sections, etc. shall be used where possible to clearly and concisely present the findings and recommendations. The Final Geotechnical Engineering Report must include all the geotechnical related contract documents, including: subsurface exploration plan, profiles and cross-sections; details; and special provisions.

1.5.6 Subsurface Findings and Parameters Report - Optional

For large projects and/or projects with complex subsurface conditions, and particularly where the subsurface exploration covers multiple design and construction sections, it is best to prepare a Subsurface Findings and Parameters Report prior to preparing the Preliminary and/or Final Geotechnical Report. Both field and laboratory explorations and testing are included in the Subsurface Findings and Parameters Report. The data (borings and testing) is presented by design/construction section, and a brief analysis and summary of findings is provided. The summary also includes any recommendations where additional exploration may be beneficial or required, based upon analysis of the data obtained from exploration and testing. Following the summary, recommendations for soil and rock design parameters are presented. Justification for the proposed parameters is included, so that a basis of acceptance exists for the Department. With all subsurface and testing information presented and analyzed, and soil and rock design parameters accepted, design work can proceed directly into final design for all sections necessary.

This approach of preparing a Subsurface Findings and Parameters Report enables the Department and its partners to assess conditions along the entire alignment, and finalize soil and rock design parameters for the alignment, prior to moving into final design. Approval of the parameters by the Department prior to performing the analyses should eliminate or significantly reduce rework, since rework is often the result of differing opinions on soil parameters. Additionally, on large projects where numerous people, and possibly various consultants, are performing geotechnical analyses, approval of the parameters prior to performing the geotechnical analyses/calculations will save time and should yield more consistent results compared with individual engineers/consultants developing parameters.

When the need for this report exists, it must be identified during project scoping so adequate time and budget can be allotted. If the need for this report is not realized until after scoping, it should be discussed with the DGE and PPM.

There is not a required format for this report, so the format should be agreed to and documented in the scope of work. At a minimum, this report should include the following:

- Subsurface Exploration Plan
- Typed boring logs
- Summary of laboratory test results and individual test reports
- Field test results (CPT, vane shear, etc.)
- Tabulation of recommended soil and rock design parameter.
- Justification of recommended parameters

1.5.7 Structure Foundation Report

The Structure Foundation Report is required for each structure on a project. This report provides foundation recommendations and construction considerations for the proposed structure. These reports are initiated, prepared and submitted after Type, Size and Location (TS&L) approval for bridges and culverts, but are required to be submitted with the TS&L for retaining walls and noise walls. The report requirements discussed in this section include the requirements set forth in DM-4, Section 1.9.4.3.1 for a Standard Foundation Submission.

1.5.7.1 Submission Format and Content

The required format and content of the Structure Foundation Report is as follows:

Cover Page
Table of Contents
[1.0](#) Introduction
[2.0](#) Summary of Foundation Recommendations
[3.0](#) Site Geology
[4.0](#) Subsurface Exploration
[5.0](#) Laboratory Testing Program
[6.0](#) Foundation Design Considerations, Analyses, and Recommendations
[7.0](#) Construction Considerations
[8.0](#) References
[Tables](#)
[Figures](#)
[Appendices](#)

Cover Page – The cover page must contain the following information:

- Title: Structure Foundation Report
- Structure Description (e.g., S.R. ____ over ____)
- State Route and Section Number
- Name of County, Pennsylvania
- Prepared for: Pennsylvania Department of Transportation
Engineering District __-0
- Prepared by: Design Consultant Name
- Month and year report submitted
- ECMS Number
- BMS Number

Table of Contents – All pages in the report must be numbered.

1.0 Introduction – Provide a brief description of the project, including:

- location of the project

- new or replacement structure
- type of structure and brief description (i.e., length, width, number of spans, beam type, wall type, wall length, wall height, etc.)

In addition to a brief verbal description of project location, provide a Project Location Map either embedded in the report or as a separate figure. The figure must show contours and surface features (streams, rivers, strip mines, etc.) and be scaled to concentrate on the project area. Ideally, use the Project Features Map that was included in the Subsurface Exploration Planning Submission prepared for the structure.

2.0 Summary of Foundation Recommendations – Provide a brief, concise list of foundation design and construction recommendations discussed in Sections 5.0 and 6.0 of the report. This summary, or something similar, should also be included on the Foundation Plan and Elevation drawing(s).

3.0 Site Geology – Provide a brief description of the geology of the project area based on published information. This information and any associated map(s) needed should be obtained directly from the Subsurface Exploration Planning Submission that was prepared for the structure. Information in this section must include:

- Geologic formation(s) and general bedrock descriptions, including: type, color, bedding, weathering, jointing and faults. Generally, a map is not required. However, if geologic features are significant to the structure foundation provide a map(s) as needed.
- Engineering geology. Provide general information, including: ease of excavation, drilling rate, and cut slope and foundation stability.
- Pertinent geotechnical findings from published reports and maps (e.g., mining, sinkholes, landslides, acid bearing rock, etc.). Provide map(s) as needed.
- If applicable, provide a stratigraphic column, for example when coal seams are pertinent to foundation design.
- Existing obstructions, or potential for encountering obstructions during pile driving operations.

Whenever practical, a tabular format is to be used when describing geology.

4.0 Subsurface Conditions – Provide a description of the subsurface exploration(s) performed for the proposed structure. Information in this section must include:

- Type of investigation(s) performed (i.e., borings, geophysics, CPT, etc.).
- Total number of borings and CPT's, coverage area of geophysics, etc. A Subsurface Exploration Plan showing the locations will be provided in Appendix A of the report.
- Description of instrumentation installed (i.e., piezometers, inclinometers, etc.)

- Contractor that performed the investigation(s) and the date(s) they were performed.
- Brief description of the general/typical subsurface conditions encountered. A detailed description is not required. Include subsurface profiles, structure borings and typed engineer's field boring logs. Depending upon the number of borings and the uniformity of the subsurface, it may be possible to describe the typical conditions of the entire project area. Where the investigation includes numerous borings and/or the subsurface conditions are not uniform, it may be more beneficial to summarize based on individual or groups of substructures. Include typical soil types, SPT N-value range and average, bedrock type, and range and average of rock core recovery and Rock Quality Designation (RQD).
- Description of significant irregularities encountered, like voids, soil filled seams, coal seams, etc.
- Existing obstructions, or potential for encountering obstructions during pile driving operations
- Summary of water levels observed in the borings.
- Summarize any other information obtained from CPT's, geophysics, instrumentation, etc.

5.0 Laboratory Test Results – Provide a description of the laboratory tests performed and a summary of the results. A detailed description of the results is not required. The individual, detailed laboratory test reports will be included in an appendix of the report along with any necessary summary tables. Include photos of unconfined compression test specimens after failure.

6.0 Foundation Design Considerations, Analyses, and Recommendations – Provide foundation recommendations for the proposed structure, including the information required by DM-4, Section 1.9.4.3.1, and provide **justification** and **discussion** of the results of analyses to support these recommendations. Include soil and rock parameters to be used in design, with calculations and references. Include cost estimates/comparisons when appropriate. As applicable include:

- Bottom of footing, pile and drilled shaft tip elevations for each substructure
- Bearing material of foundation
- Minimum footing embedment depth/frost penetration depth
- Bearing resistance for spread footings on soil/rock
- Pile type, size, tip reinforcement and resistances (strength and service limit states)
- Pile driving criteria (i.e., Case 1, 2, etc.)
- Drilled shaft diameter(s) and resistances (strength and service limit states)
- Estimation of foundation settlement
- Scour depth and countermeasure recommendations
- Corrosion recommendations
- Pile downdrag potential and mitigation
- Seismic design parameters and liquefaction potential

- For bridge replacement or widening projects, information on existing foundations (i.e., type of foundation, footing and pile/shaft tip elevations, etc.) and performance of existing foundations (i.e., signs of settlement, instability, etc.).

7.0 Construction Considerations – Provide construction related considerations and recommendations for the proposed structure. Additionally, provide justification and discussion to support the recommendation. As applicable include: dewatering, blasting, shoring/parameters, existing foundations, pile monitoring (PDA), vibrations, re-driving, overexcavation/class C, foundation notes (plans or specifications), obstructions (known or potential) during pile driving operations, required geotechnical involvement during construction.

8.0 References - It is preferred that references used to prepare the report/recommendations be included directly within the report text. A formal listing is not necessary. If references cannot be included in the text provide a list of the references in this section of the report. Any information from these references relevant to the justification or recommendations provided in the report that cannot be readily incorporated into the body of the report must be included in the figures section or an appendix as appropriate.

Tables - Include tables that are not or cannot be embedded in the text of the report. This includes summary tables of material that may also be presented in the appendices. For example, a summary table for laboratory test results would be presented in this section while the individual test results and work sheets would be presented in the appendices.

Figures - Embed figures directly in the report when possible and appropriate. Place figures that cannot be easily embedded within the report text in this section. Only include figures that are “value added”, and include as much information as possible on the same figure. The following figures are required in this report and most likely can be obtained from a previously prepared report (e.g., Subsurface Exploration Planning Submission), although some modifications/additions may be required:

1. Site Location Map – As discussed in the Introduction section of this report, this figure must be a large enough scale so that someone not familiar with the site can locate it.
2. Project Features Map – This figure must be scaled to focus on the immediate project area. This figure must include topographic contours and surface features (streams, rivers, roads, strip mines, etc.).
3. Geologic Map

Other “value added” figures shall also be included in this section of the report. These figures may include: Maps from Open File Reports (i.e., sinkhole maps), Mine Maps, Landslide Maps, Acid Bearing Rock Map, etc.

Appendices

Appendix A – Subsurface Exploration Plan and Structure Borings

Appendix B – Typed Engineer’s Field Boring Logs

Appendix C – Driller’s Logs

Appendix D – Core Box Photo Logs

Appendix E – Full Laboratory Test Reports*

Appendix F – Subsurface Profiles and Calculations/Analyses

Appendix G – Geotechnical Details and Special Provisions

Appendix H – Applicable Quality Assurance Forms:

Foundations (Form D-505)

Substructures (Form D-514)

Integral Abutments (Form D-515)

Proprietary Retaining Walls (Form D-516)

Flexible Retaining Walls (Form D-517)

* Provide individual laboratory test reports for each test performed. A Professional Engineer licensed in the Commonwealth of Pennsylvania must certify that all testing was conducted in a laboratory accredited by the AASHTO Materials Reference Laboratory (AMRL) for the individual test methods conducted.

1.5.7.2 Foundation Submission Letter

A foundation submission letter must accompany the report and include the information indicated in DM-4, Section 1.9.4.3.1. It may be advantageous to use a table or tables to present the required information in a concise, comprehensible manner. This submission letter from the design consultant to the Department shall note the contractual requirements with respect to the report and the status of the report (i.e., is it a draft or final report, are any addendums anticipated, etc.). The report shall be presented on standard letter sized paper. Plans, profiles, cross-sections and figures that are not legible on standard letter sized paper must be presented on 11x17 foldout sheets. Bullet points, tables, figures, sections, etc. shall be used where possible to clearly and concisely present the findings and recommendations.

1.5.8 Foundation Design Guidance Report

A Foundation Design Guidance Report is required for Design/Build projects where an “As-Designed” foundation is not provided by the Department, in which case the Design/Build team (contractor and engineer) is responsible for preparing the Structure Foundation Report. The purpose of this report is to provide the results of any subsurface exploration and recommendations of permissible foundation types to the Design/Build teams. Requirements for this type of report are included in Publication 448, Chapter 3, Section 3.2.3.1.

1.6 GEOTECHNICAL INVOLVEMENT DURING CONSTRUCTION

Prior to construction, it is important to consider whether involvement from Department and/or consultant geotechnical staff during construction is necessary and/or beneficial. The need for involvement may be dictated by the requirements of a special provision, such as when geotechnical related items control the flow or sequence of work (e.g., quarantine requirements for settlement or slope stability). The need for involvement could also be dictated by a critical or complex geotechnical issue that the DGE deems is necessary to review and/or confirm construction activities. If geotechnical staff involvement is deemed to be necessary or potentially beneficial, appropriate provisions must be made to arrange for these services. If a consultant is needed, a scope of work must be developed and included in a post design/construction services agreement. If PennDOT geotechnical staff is to be used, the PPM and DGE must also develop a scope of work to ensure that in-house staff has the capability and availability to perform the anticipated work.

1.6.1 Requirements for Geotechnical Specialist During Construction

During the design phase, constructability issues should have been considered when developing design recommendations, including the geotechnical recommendations. Although constructability is considered during design, geotechnical staff involvement is often beneficial or crucial during construction for a variety of situations or conditions reasons, some of which are discussed below.

1.6.1.1 Unforeseen Subsurface Conditions

Although borings are performed during the design phase to estimate subsurface conditions, in actuality, these borings sample only a miniscule amount of soil and rock underlying a project site. When operations occur during construction, like fill placement, soil/rock excavation, shallow foundation exploration in difficult geology, deep foundation installation, ground improvement, etc., subsurface conditions differing from those anticipated from the borings are sometimes encountered or claimed to have been encountered. When such situations occur, geotechnical staff may be needed to verify a change in conditions and if necessary modify recommendations made during the design phase, provide recommendations for previously unanticipated conditions, or provide related guidance to facilitate the continued flow of construction activities (i.e., keep the project moving).

While it is generally not anticipated if unforeseen or differing subsurface conditions will be encountered, the potential for encountering unforeseen or differing conditions should be considered. The nature of some geologic settings does increase the potential for encountering unforeseen, differing or difficult subsurface conditions. Overall project subsurface conditions are difficult to predict from a collection of individual borings. This may be the case, for example, when the bedrock is pinnacled, such as in limestone and dolomite formation, in complex geologic settings, or in areas of past mining activity. Also, if obstructions such as cobbles and/or boulders are encountered in borings, it is difficult to predict the actual size and quantity/volume of the obstructions underlying the site during design. As a general rule of thumb, the more complex the subsurface conditions and proposed construction are, or the greater

the level of past human activity (e.g., mining, excavation, filling, etc.) the more likely it is that subsurface conditions interpreted from the borings during design may vary from or be more complex than the subsurface conditions encountered during construction.

1.6.1.2 Geotechnical Instrumentation

Geotechnical instrumentation, including piezometers, inclinometers, settlement plates/sensors, earth pressure cells, etc., is sometimes used on projects to estimate the response of the subsurface to loads from embankments and structures to verify design assumptions, or to provide necessary control of the nature or pace of construction activities. While many areas of geotechnical engineering are, by nature of the variability of subsurface materials, addressed with a deliberately low level of precision (and therefore appropriate levels of conservatism, factors of safety or load and resistance factors), for geotechnical instrumentation to be effective, an exactly opposite approach must be taken for its planning and execution. A deliberately high level of precision and care must be practiced to counter the variability and unknowns of subsurface conditions, if the results of the instrumentation are to be of any value to the project.

The importance and necessity of a well planned and executed geotechnical instrumentation plan cannot be understated. This includes:

- Understanding the subsurface conditions and nature of the construction
- Understanding the need and purpose for the instrumentation
- Planning for both the types and layout of instrumentation
- Proper design of the instrumentation data acquisition system
- Proper installation
- Adequate maintenance and protection of components
- Actual data acquisition
- Interpretation of the data to address the purpose or problem the instrumentation was intended to address
- Adequate redundancy of instruments and instrument types
- Inclusion of necessary restrictions and/or construction control in contract specifications.

A poorly planned and/or executed instrumentation program can, at the very least, result in inefficient construction, an inefficient design, or unverified construction quality or adequacy, but much worse can result in construction delays, claims, failures, or possibly injury or death.

When instrumentation is used, several associated tasks typically must be completed. These tasks and requirements are best suited for a geotechnical specialist. Some typical instrumentation tasks include:

1. Review of submittal(s) - A submittal is required by the contractor, which must be reviewed by a department representative, to identify the specific instrumentation that is proposed to be used. Project specifications must require proof that all instrumentation is properly calibrated and fully functional prior to installation. Project specifications must address and provide requirements for the maintenance,

care and protection of all instrumentation, and specify replacement requirements, penalties and recovery plan in the event of damage or non-functioning instrumentation. The equipment proposed must meet project specifications and needs.

2. Instrumentation installation - The instrumentation must be installed/ constructed. The project specifications must provide detailed installation and performance requirements, personnel experience and requirements, the needed qualifications of personnel to inspect all installations, and the requirements for verifying proper operation and function.
3. Data collection - Data must be collected from the instruments. Collecting data from instruments can range from physically using a probe, such as an inclinometer or water level indicator, to downloading data from a website when a remote/wireless collection system is used, such as from a vibrating wire instrument. The project specifications must provide the requirements and experience of personnel collecting the data, and requirements for frequency and method of data collection, storage of data, and transfer of data.
4. Data interpretation - The collected data must be compiled and reviewed/interpreted, and the project specifications must provide criteria for if, when and/or how to proceed with construction. For example, pore pressure from piezometers and/or settlement from settlement monitors must be evaluated to determine if embankment can be placed. Project specifications must require that a geotechnical specialist with the required experience and knowledge, and that was involved or fully familiar with the design, be involved with interpreting and evaluating the data from the instruments. The project specifications must indicate the required qualifications and experience of the geotechnical specialist.

1.6.1.3 Ground Improvement

Ground improvement techniques, including compaction grouting, jet grouting, stone columns, wick drains, vibro-compaction, lime/cement stabilization and others, are becoming more widely used on projects. Successfully completing this work is highly dependent upon preparation of an organized, complete and detailed specification that addresses both necessary operational requirements and performance criteria. The specification must be adhered to and should follow generally accepted industry practices. The specification must require that a geotechnical specialist either involved with the design or thoroughly familiar with the design and the ground improvement system and technology specified, oversee the work during construction. When ground improvement is used, several tasks typically must be completed during construction. **These tasks must be performed by a geotechnical specialist.** Typical tasks include:

1. Review of submittal(s) - A submittal is required from the contractor, which must be reviewed by a Department representative, to identify the specific materials, equipment, procedures, etc. that will be used to ensure that they meet the specifications.
2. Oversight of the test section - It is required that the contractor perform a test section to demonstrate that the proposed materials, equipment, procedures, etc.,

function and perform in a manner that will satisfy project goals and requirements, and meet all performance requirements. If satisfactory results are not obtained from the test section, the contractor must propose changes that are acceptable to the Department, and perform a new test section demonstrating satisfactory performance.

3. Inspection and monitoring - The ground improvement “production” work must also be closely inspected and monitored, to assure that the approved materials, equipment, procedures, etc. are being used, and to provide accurate and thorough documentation. Monitoring and documentation are critical to successfully completing ground improvement projects.
4. Performance testing - All ground improvement techniques must require performance testing to assure that the necessary and required improvement to the subsurface was achieved. This performance testing must be observed by a Department representative. The results of the performance tests must be evaluated by a Department representative to determine if the ground improvement work meets the intended goals, or if additional ground improvement work is necessary.

1.6.1.4 Deep Foundations

Deep foundations, including driven piles, micropiles and drilled shafts are commonly used to support structures. Installation of these foundations should always be observed by a Department Representative, but in some cases this oversight should be provided by a geotechnical specialist. Situations that warrant oversight by a geotechnical specialist are discussed below.

1.6.1.4.1 Soluble Bedrock

Deep foundations bearing on or in soluble bedrock, like limestone, dolomite, and marble often have extreme variation in the top of rock surface, so additional care is needed during installation to evaluate the required pile or shaft tip elevation. A geotechnical specialist may be better suited to compare observations made during construction with subsurface information and design assumptions to determine the adequacy of the installed foundations.

1.6.1.4.2 Obstructions

Deep foundations may be installed in subsurface conditions containing obstructions, like cobbles, boulders, and demolition debris. Encountering obstructions with individual deep foundation elements is generally very unpredictable, even when the results of the subsurface exploration indicate the potential for encountering obstructions. When obstructions have been encountered during the design subsurface exploration or are otherwise suspected, it is important to ensure that foundation elements are extended to competent bearing material and not founded on an obstruction. In situations where obstructions are indicated as a possibility or otherwise suspected, it is important to make sure provisions have been included in the contract to address this potential. This may include predrilling requirements to get past the obstructions, or exploration of the subsurface at each foundation element prior to installation, to check for

obstructions. Such investigations are usually performed with small diameter, rapid penetration drilling equipment without sampling (e.g., air-track drilling). Additionally, encountering obstructions during installation of deep foundations is often costly and may result in contractor requests for additional compensation (e.g., claims, change orders, etc.). When the potential for obstructions is indicated or suspected, it is important to have detailed records of installation procedures, observations during installation, etc., from a trained geotechnical specialist to aid in evaluating compensation request(s) from the contractor.

1.6.1.4.3 Air Track Drilling

Deep foundations that utilize air track drilling to help establish tip elevations, rock socket lengths, etc. Air track drilling is often used during construction to help estimate the top of bedrock when the design subsurface exploration revealed “extreme” variations in the top of rock, and to help identify the location of voids, soil seams, weak rock (claystone) below the top of rock. It is important that a trained geotechnical specialist, who is familiar with and understands the geologic setting and the proposed construction observe and document the air track drilling and foundation installation to better ensure that competent and adequate foundations are constructed.

1.6.1.4.4 Drilled Shafts

Drilled shafts typically carry large axial and/or lateral loads, and often very few shafts are used to support a single bridge substructure. Therefore, unlike a typical driven pile foundation, there is often little redundancy when drilled shafts are used. Each shaft carries a very high portion of the total load. Thus, any undetected problems in any one shaft will present a significant potential risk. Consequently, it is critical that drilled shafts be installed to the proper depth, and that appropriate means and methods be used to install them. A geotechnical specialist, who is familiar with the design, geologic setting and the proposed construction, can help assure that shafts are constructed in competent materials that provide the resistance required to satisfy design needs. Drilled shafts require a rock socket to carry axial and/or lateral loads, and a geotechnical specialist should evaluate rock socket conditions to assure that all design assumptions are valid. Drilled shafts also often require special testing during and after construction, including cross hole sonic logging, mini SID (submerged inspection device) testing, sounding of shaft bottom, drilling (coring or destructive) below the bottom of the shaft, testing of drilling slurry (typically polymer), etc., that the geotechnical specialist must be qualified to assess.

1.6.1.4.5 Micropiles

Micropiles used on Department projects typically require a rock socket to provide axial and lateral resistance, and micropiles are often used where voids, soil seams, weak rock layers, etc., underlie a project site. It is therefore important that the micropile rock socket be installed in adequate bedrock to support the proposed loads. Determining the adequacy of the rock socket is typically based on the response of the drill rig (i.e., rate of drill stem advancement, down pressure, water pressure, sound, etc.) and the type of cuttings that exit the drill hole. The drill operator is often relied upon heavily to evaluate these conditions due to their experience

installing micropiles. For proper QA assessment, a geotechnical specialist should observe and document all conditions during installation, communicating with the drill operator to assure that an adequate, suitable rock socket is obtained for foundation installation. With close observation of the drilling, the geotechnical specialist can prevent excessive pile lengths (i.e., reduce cost) and can track quantities, depths and/or socket lengths necessary to satisfy design requirements, and fully document pile installation (including item quantities for quality control and assurance, or for payment calculation).

1.6.1.4.6 Friction Piles

Friction piles generally require more judgment to determine when adequate bearing resistance is achieved compared to piles driven to refusal. A Pile Driving Analyzer (PDA) is typically used with the test pile(s) to help set the end of driving criteria. However, due to natural variations in subsurface conditions, assessment for pile capacity is not always as simple as following prescribed PDA criteria during production pile driving. With review of PDA records, available boring information, and close observation of pile driving operations, a qualified geotechnical specialist can help assure adequate capacity in difficult or unusual pile driving conditions.

1.6.1.4.7 Augercast or Continuous Flight Auger (CFA) Piles

Successfully installing augercast piles relies heavily on observations made during construction, including grout pressure, rate of auger withdrawal, grout injection rate/volume, and more. Equipment calibration is also critical. Oversight of augercast pile installation should be provided by a geotechnical specialist familiar with the subsurface conditions, the design requirements and the proposed construction. The geotechnical specialist must have experience and knowledge with the installation of augercast piles in order to help assure the quality of construction, and to assure that the final product will function as required and provide the required capacities.

1.6.1.4.8 Contractor Submittals

Contractor submittals are required for numerous deep foundation types, including micropiles, drilled shafts, and augercast piles. These submittals are required to document contractor experience, proposed materials and methods to be used, load testing and QA/QC procedures, and more. Review of these submissions by a geotechnical specialist having knowledge and experience with the proposed construction, the specific design, and the site subsurface conditions is important to make sure that the planned construction is adequate, design requirements are satisfied, and that the quality and adequacy of the product is verifiable.

1.6.1.5 Acid Bearing Rock (ABR)

Acid bearing rock exposed in project cuts for roadways and/or structures can cause extreme environmental problems and prove to be very costly. Projects where design subsurface explorations identify acid bearing rock, or where published information or past experience indicates acid bearing rock may be present, should use a qualified geotechnical specialist to

observe excavation activities. Identifying acid bearing rock quickly is imperative to prevent or minimize negative environmental impacts, and prevent or mitigate impacts to the project. If the project is one where encountering ABR is anticipated, the qualified geotechnical specialist can assure that the planned mitigation is adequate and properly executed. If it is a project where the potential for encountering ABR exists, then the qualified geotechnical specialist can provide the oversight necessary to identify if such materials are encountered, assist in developing a mitigation plan to address the problem, and assure that the mitigation plan is properly executed, while assisting to minimize impacts to the project.

1.6.1.6 Voids

Voids beneath a project site, either naturally occurring from the solutioning of soluble bedrock (i.e., limestone, dolomite, marble) or manmade from mining, will require treatment if they are known or anticipated to impact the proposed construction, or have an impact on long term performance. Whether to treat voids will typically be determined during the design phase, although often times the exact extent/limits of the void are not known at this time. Additionally, voids not identified by the design subsurface exploration may be encountered during construction. Whatever the situation, the geotechnical specialist can provide assistance and a level of assurance that:

1. The locations of anticipated voids are properly identified
2. The existence and extent of the anticipated voids are verified
3. Proposed mitigation of anticipated voids is properly monitored and executed
4. Unforeseen voids are identified and delineated
5. Effective mitigation proposals for the unforeseen voids are developed
6. Mitigation of unforeseen voids is properly monitored and executed
7. Quantities required for payment are tracked
8. Voids are adequately filled, and verification of such is provided (assuming means or measures exist to assess the adequacy of void filling operations).

1.6.1.7 Landslide Remediation

Landslide remediation projects are often challenging since the construction work has to be performed within an unstable area, and there are frequently other constraints (such as maintenance of traffic or right-of-way restrictions) that control or limit stabilization options. These projects commonly:

- Require adherence to a strict construction sequence
- Have restrictions on stockpile locations, haul road locations, working in adverse weather conditions, length of excavation limits and more
- Include instrumentation such as slope inclinometers and piezometers
- Require judgment to determine if modifications to contract plans or control of construction activities are required due to observations made or conditions encountered during construction
- Require careful monitoring of subsurface conditions and slope movement during remediation activities.

This work should be conducted under supervision of a qualified, on-site, geotechnical specialist, to assure that the mitigation plan is properly executed and constructed. The on-site geotechnical specialist may also be able to identify potential problems with the planned mitigation, and/or provide options to either avoid such problems or adjust the plan when anomalies are encountered.

1.6.1.8 Temporary Shoring

Contractor submittals are required for temporary shoring conditions. These submittals are required to document the proposed design, materials, and methods to be used for the temporary shoring. Review of these submissions by a geotechnical specialist, either in house or by consultant reviewer, having knowledge and experience with the design and proposed construction of the temporary shoring is required.

1.6.2 Geotechnical Pre-Construction Meeting

Any project involving specialty geotechnical work, sensitive or complex geotechnical issues or conditions, significant foundation work, complex or difficult subsurface conditions, large earth structures, mitigation of geotechnical hazards, or repairs of geotechnical related failures or instabilities, should have a pre-construction meeting which includes the District Geotechnical Engineer or their representative (who is familiar with the project including subsurface conditions, the design and proposed construction activities) and project construction personnel. A geotechnical pre-construction meeting should be considered for the types of projects discussed in [Section 1.6.1](#), and possibly others. The meeting should cover any of the conditions discussed above to:

1. Provide a situational awareness and understanding of the condition(s)
2. Establish necessary relationships, channels of communication, and protocols
3. Define responsibilities, constraints and needs (of all parties)
4. Plan and coordinate any activities required during construction
5. Identify any potential weakness in coordination efforts
6. Establish protocols to address any anticipated or unforeseen problems
7. Identify specific problems or conditions that are anticipated or have a potential for developing.

Benefits of a geotechnical pre-construction meeting may include:

1. The geotechnical specialist can make the inspection staff aware of specific areas of the project that are of geotechnical concern, such as unstable ground, landslide and settlement prone areas, sinkholes, potential areas of acid bearing rock, etc.
2. The geotechnical specialist can identify geotechnical details and/or special provisions that are in the contract and describe their purpose and importance. Explaining the purpose/rational behind these will aide inspection staff during construction if questions arise.

3. The geotechnical specialist and construction inspection staff can discuss possible variations to the subsurface conditions that may be encountered and how it affects the proposed details and special provisions.
4. The construction inspection staff can ask specific questions regarding the contract plans, details and special provisions. They can also discuss any concerns with the proposed construction, and discuss ways to complete the work. The use of alternate details, methods, materials, etc. can be discussed.
5. Contact information of geotechnical and inspection staff can be exchanged so that if geotechnical related questions arise during construction they can be addressed in a timely manner.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2018
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 2 – RECONNAISSANCE IN GEOTECHNICAL ENGINEERING

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2.1 INTRODUCTION

This chapter of the publication provides guidelines, recommendations, and considerations for performing both office and field reconnaissance. Numerous references are discussed that should be used for performing office reconnaissance, and guidelines and recommendations are provided to aid in performing field reconnaissance.

The term “reconnaissance” is generally defined as a “preliminary survey to gain information”. From a geotechnical engineering standpoint, reconnaissance refers to the search of available information to gain an understanding of the topographic and geologic features of a project site. The term reconnaissance used herein refers to both office and field reconnaissance. Office reconnaissance consists of locating and reviewing topographic and geologic information, and any available geotechnical records from past projects at or adjacent to the proposed project location. Available information typically includes maps, reports, publications, and aerial photographs. Field reconnaissance consists of physically inspecting the project site to observe and document topographic and surficial geologic features.

Office reconnaissance should typically be performed before field reconnaissance. Information obtained from office reconnaissance will usually be more general in nature compared to site specific observations made during field reconnaissance. Field reconnaissance is used to further investigate significant findings identified during office reconnaissance, and to make observations that are not possible from the information reviewed during office reconnaissance.

2.1.1 Purpose

The primary and global purpose of the reconnaissance phase of geotechnical engineering is to obtain the information necessary to develop and support a subsurface investigation (and lab testing) plan that is appropriate for the proposed design and construction activities. Reconnaissance also provides a valuable source of documented existing conditions along the proposed alignment that can serve to help identify specific design and/or construction needs. One of the main objectives of reconnaissance is to gain an understanding of the subsurface conditions (soil, rock, and water) at a project site without or prior to performing a subsurface exploration (e.g., borings, test pits, etc.). Published information is almost always available to indicate the type(s) of rock that underlie a site, and oftentimes information is available to provide an indication of soil type(s) and thickness. Published data can also provide preliminary information relative to groundwater conditions. Visual observation, assessment, and identification of soil and rock in natural exposures, stream/river beds/banks, roadway cuts, excavations, etc., may also provide information on soil and rock types underlying a project site.

Another main objective of reconnaissance is to observe topographic features, both from available information and site observations. Topographic features such as terrain, landslides, vegetation, seeps, and others may provide insight on the underlying soil and rock. Additionally, site observations are needed to gain an understanding of the access and equipment needs to complete the subsurface exploration and may provide insight on project constructability.

The information obtained from reconnaissance is used for subsequent geotechnical tasks. In most cases, the information will be used to help develop the subsurface exploration program. In cases where a subsurface exploration will not be performed, for example possibly during the Alternatives Analysis or Preliminary Design Phase of a project, the reconnaissance information may be used to help formulate preliminary geotechnical recommendations.

2.1.2 Level of Effort

Reconnaissance may be required during each phase (i.e., Alternatives Analysis, Preliminary Design, and Final Design) of a project in which geotechnical involvement is needed, or it may only be needed during one phase of a project. When reconnaissance is needed, the level of effort required to adequately complete reconnaissance will vary based on the:

- Phase of the project
- Type, size, and complexity of the project
- Complexity of the topography and subsurface
- Presence of geohazards
- Amount of available information

In general, as the project advances through the various design phases, reconnaissance will become more detailed and focused since the roadway alignment, profile, structure location(s), etc., are better established. Additionally, as more geotechnical information is obtained throughout the design phases, reconnaissance should be concentrated on those areas that are or appear to be geotechnically significant.

Projects that include an Alternatives Analysis Phase, like a new roadway alignment, are generally considered to be complex. Reconnaissance performed during the Alternatives Analysis must include all the alignments being considered; therefore, although reconnaissance must be thorough, it does not necessarily have to be overly detailed because the main purpose of the Alternatives Analysis is to identify pros and cons of the various alignments to help select the preferred alignment. As the project progresses through the preliminary and final design phases with the single, preferred alignment, more detailed reconnaissance is required to aid in providing geotechnical design and construction recommendations. Projects that do not involve alternatives analysis, such as roadway widenings and/or structure replacements, may only require one phase of reconnaissance since a detailed reconnaissance can most likely be completed at the onset of the project.

2.1.3 Presentation of Findings

Relevant findings from office and field reconnaissance must be documented in the appropriate geotechnical report(s). This is very important to both provide a record of the information reviewed and relevant findings and to permit the information to be easily retrieved and used during subsequent phases of the project. Proper documentation will help ensure that important information is not overlooked and effort to perform this work is not duplicated during a latter phase of the project. A detailed discussion of these requirements is provided in [Chapter 1](#) of this publication.

It is most useful to present relevant features identified during office and field reconnaissance on project specific mapping. This presentation helps to better understand the location of the features with respect to the proposed project and potential impact of the various conditions and gain a “global” sense of project conditions and needs. For example, it is useful to show bedrock formation contacts, faults, anticlines/synclines, landslides, and other features that are or may be relevant to the geotechnical design and/or construction as they relate to one another. Features that cannot be presented on project mapping must be shown such that their location to the project can be determined. Relevant information that cannot be shown on mapping must be summarized in the text of the report.

2.2 OFFICE RECONNAISSANCE

Office reconnaissance should typically be performed prior to field reconnaissance. Office reconnaissance commences with a search for available information on topography, soils, and geology. Available information must be reviewed, and relevant/potentially significant geotechnical information should be further investigated during field reconnaissance. Included in this section are common sources of geotechnical related information. These sources, which include maps, publications, reports, aerial photographs, and others, should be used to perform office reconnaissance. Note that sources and links presented below were current as of May 2016; however, it is likely that at least some of these sources/links will change over time.

2.2.1 Project Plans, Profiles, and Cross-Sections

Any available project specific plans, profiles, and cross-sections must be reviewed. This may include information from previous projects at the site. These not only have to be reviewed to gain an understanding of the project, but they may reveal information important to the geotechnical aspects of a project. Landforms and terrain (i.e., geomorphology) are the first indicators of subsurface conditions. If project specific mapping with contours is available, it should be studied carefully for information it can offer in assessing geotechnical needs and concerns. For example, steep terrain could be an indication of colluvial/talus slopes, shallow bedrock, bedrock outcrops, or landslide potential. Hummocky and/or downslope “bulging” terrain could also be an indication of landsliding. Flatter terrain may be an indicator of weak soils that are not able to support “steep” slopes, and may be an indicator of soft soils.

Review of the project specific plans also provides an indication of areas where subsequent field reconnaissance should be concentrated. These areas include: deep cuts, high fills, structure locations, steep terrain, low lying/wetland areas, and areas of irregular topography. However, regardless of the topographic features or the proposed construction, field reconnaissance must be performed for all areas of the project because topographic features are not necessarily indicative of subsurface conditions.

2.2.2 Project Specific Geotechnical Information

Once the proposed alignments are reviewed, project specific geotechnical information, if available, must be obtained and reviewed. Consultants must request this information from the

Department. Project specific geotechnical information will most likely not exist for projects that involve new alignments; however, there may be useful information from nearby roadways and/or structures. Existing geotechnical information should exist for roadway widening/improvement projects and for structure widening/replacement projects. Project specific information could include geotechnical reports, roadway subsurface profiles, structure boring tracings, design drawings, boring logs, and as-built plans. The Department’s gINT database will be a primary source of geotechnical information as it becomes populated with gINT project files. However, as of 2016, only Central Office and the Districts have access to the Department’s gINT database. Therefore, Districts can provide relevant gINT project files to Business Partners, or Business Partners can request that Districts provide any existing relevant gINT project files for reference. Project specific geotechnical information will be more detailed compared to information obtained from published sources and is therefore very important to obtain and review. This information is not only important to review to help prepare the subsurface exploration program, but it also may be of sufficient quality/detail to reduce the scope of the subsurface exploration program. Care should be taken when assessing existing subsurface information as standards for data collection have evolved over time. It is important that the procedures and practices used to obtain the information are identified and understood.

2.2.3 Topographic Maps

As previously discussed, topographic maps can provide very useful geotechnical information. Topographic maps include contours at various intervals depending on the slope of the terrain and scale of the map, and these maps show natural and cultural features. Topographic maps also can be used to evaluate site access, which is important when planning field reconnaissance and subsurface explorations as well as constructability as a project evolves. These maps should be one of the first items reviewed during office reconnaissance.

Topographic maps were first published by the United States Geological Survey (USGS) in 1882. Most USGS map series are divided into quadrangles bounded by two lines of latitude and two lines of longitude. For example, a 7.5-minute quadrangle (i.e. quad) map, covers an area of 7.5 minutes of latitude by 7.5 minutes of longitude. Note that, although quad maps appear to cover a rectangular area, they are not exactly rectangular due to the curvature of the earth’s latitude and longitude lines. Common published scales for USGS quad maps in order of decreasing detail are provided in the table below.

Quadrangle Size	Scale	Equivalent Representation in Map Distance
7.5 minute	1:24,000	1 inch = 2,000 feet
15 minute	1:62,500	1 inch = approx. 1 mile
30 x 60 minute	1:100,000	1 inch = approx. 1.6 miles
1 degree x 2 degree	1:250,000	1 inch = approx. 4 miles

USGS topographic maps, which include maps that cover Pennsylvania, can be downloaded free of charge from the following USGS link: [Downloadable USGS Topographic Maps](#).

Project specific mapping is typically done to a large scale of 1:600 (1" =50'), which is much more accurate than published topographic maps. However, sometimes smaller scale mapping can provide a better overall "picture" of the project site, and can reveal information that larger scale mapping cannot. For example, landslides and/or unstable slopes, alluvial deposits/fans, and sinkhole features are sometimes more apparent on smaller scale mapping. Published mapping is also advantageous to review because mapping is typically available from various time periods (e.g., decades or centuries). Review of mapping from various time periods can indicate past land usage and reveal features that are no longer visible or apparent. For example, over time, stream/river channels may have meandered or filled in, railroads may have been abandoned, and mine workings (e.g., deep mine opening, strip mine, mine spoil, etc.) may have been reclaimed. Additionally, over 5,000 historical topographic maps covering areas of Pennsylvania are available online from the USGS using the link: [Historical Topographic Maps](#). These maps may also be useful for investigating past land uses and changes in topographic features.

Topographic maps are also important to review because they can provide an indication of geotechnical items that may require further/careful exploration. For example, ground contours provide extremely useful information. A project site located in an area of closely spaced contours indicates steep terrain. Steep terrain typically indicates the presence of colluvium, relatively shallow bedrock, and/or bedrock outcrops. The soils on these steep slopes are often at their natural angle of repose; therefore, cut slopes excavated steeper than the natural slope angle may not be stable, and surcharge loads (e.g., embankment fill, pavement, or structure) placed on these slopes may not be stable without special geotechnical treatments. Irregular, hummocky, and downhill sloping/bulging contours could also be an indication of landsliding. Conversely, a project site containing widely spaced contours indicates flat or gently sloping terrain. Flat terrain could be a sign of past landsliding due to underlying weak material, and it can be an indication of soft, wet soils.

2.2.4 Aerial Photography and Imagery

Aerial photography is the process of taking photos of the ground using a camera that is typically mounted to a manned or unmanned aircraft (e.g., plane, helicopter, drone, etc.). If unmanned aerial surveillance or drones are used for aerial photography, they must maintain compliance with all federal, state, and local regulations regarding operations and use. Aerial photography is another source of topographic and geologic information that is extremely useful during office reconnaissance and should be reviewed for new roadway alignment projects and landslide remediation projects. Roadway widening and structure replacement projects may not benefit from the review of aerial photography, and therefore, it is the DGE/PGM's responsibility to determine if review is necessary. Also, note that satellite imagery of a project site is commonly available to view, and can be used in the same manner as aerial photography.

2.2.4.1 Non-Rectified Aerial Photography

Non-rectified aerial photographs have not been corrected for camera tilt, lens distortion, changes in terrain elevations, and flying height. These photographs cannot accurately be used to measure true distances; however, they are still useful in reviewing project history and features.

The Pennsylvania Geological Survey (PGS) Library maintains collections of hard copy non-rectified aerial photographs from 1946 to 1999. These photos must be viewed at the PGS library located in Middletown, PA. The photographs are not available electronically. These photographs can be checked out for up to one month, and appointments for obtaining them may be scheduled online using the link: [Calendar for PGS Library Appointments](#).

The PGS collections are from the U.S. Geological Survey and U.S. Department of Agriculture (USDA). Coverage area is statewide and photograph scales vary from 1:20,000 to 1:80,000. The photography includes both leaf on and leaf off (i.e., the presence or lack of foliage), most are black and white, but some of the photographs are color infrared. The PGS collection of aerial photographs consists of four (4) series, and they include:

- Agricultural and Stabilization Conservation Series (1946-1981)
- U.S. Geological Survey (mostly late 1960's and 1970's)
- National High-Altitude Program (NHAP, initiated in 1980)
- National Aerial Photography Program (NAPP, late 1980's thru 1999)

An online source for non-rectified aerial photography is at Penn Pilot which is hosted by the Pennsylvania State University, at the following link: [Penn Pilot](#). The PGS had thousands of historical (1937–72) aerial photographs from the Agricultural and Stabilization Conservation Series scanned and put online.

The USGS offers their aerial photography and other imagery on their Earth Explorer website at the following link: [Earth Explorer Aerial Photography](#). Some low-resolution images may be downloaded for free. However, users are charged a fee for higher resolution images.

One of the limitations of non-rectified vertical aerial photography is the lack of apparent relief. To view aerial photography in three dimensions, a mirror stereoscope is used to look at stereopairs of photographs. A stereopair consists of a pair of photos with sufficient overlap (typically 60%) between successive photos. Ideally, the review of aerial photography using a stereoscope should be performed by a trained individual, typically a geologist with experience viewing aerial photography. However, even an untrained eye can detect important features from aerial photographs.

When reviewing aerial photographs, it is best to obtain photographs taken during as many different time periods as possible. Review of photos from different time periods may reveal important features which may not be visible at present time. This could include excavations from small quarry or mining operations that have been backfilled (possibly with waste material/debris), or buildings that have been demolished but still have foundations in place. Review of photos may also show changes in the topography and vegetation, which can be an indication of the presence of seeps/water or landsliding. Tonal variations (changes in the color or shade of the topography) can be an indication of dense vegetation (darker shades) due to the presence of water, and tonal differences can be the result of abrupt changes in topography due to landsliding (e.g., head scarp or toe bulge). Changes in stream/river location can also be observed.

Site specific aerial photography provided by commercial aerial-photography companies can also be useful to complete the project reconnaissance if available and/or necessary.

2.2.4.2 Digital Orthorectified Aerial Photography

Orthorectified aerial photography, which is commonly termed digital orthophoto, digital orthophotograph, or digital orthoimage, has been geometrically corrected or “orthorectified” such that the scale is uniform and the photo has the same lack of distortion as a map. The digital photo has been corrected so that its pixels are aligned with the longitude and latitude lines and have small defined coverage. Orthophotos can be used directly to measure true distance because the photo has been adjusted for topographic relief, lens distortion, and camera tilt. As part of office reconnaissance, a good preliminary project base map consisting of superimposed topographic contours on a digital orthophoto base should be obtained. Digital orthophotography can be accessed at several online sources. A few of these sources are listed below:

- Pennsylvania Spatial Data Access (PASDA) is the official public geospatial information clearing house for the Commonwealth of Pennsylvania. The Pennsylvania Imagery Navigator (PIN) has been developed by PASDA for fast and easy access to digital orthophotography and other imagery. The PIN can be found online at the following link: [Pennsylvania Spatial Data Access Maps](#).
- Another source for aerial photography is the Department’s Photogrammetry Asset Management System (PAMS) at the following link: [PAMS Photography](#). This site is for a geographic search for available aerial photography as well as traditional survey data (ground control and bench mark information).
- 7.5 minute digital orthophotos covering Pennsylvania with topographic contour overlay can be downloaded from the USGS at the following link: [USGS 7.5 Minute Digital Orthophotos](#)

2.2.4.3 LIDAR Imagery

Light Detection, and Ranging (LiDAR) is a surveying method that uses laser light to measure distance and create a three-dimensional image. LiDAR can be used to survey the ground surface or structures, and LiDAR data can be collected using ground based or airborne equipment. Due to the large number of data points that are collected, LiDAR can be successful at penetrating vegetation to accurately survey ground surface. However, in heavily vegetated areas, such as coniferous areas, LiDAR survey may not be successful. A LiDAR data set for the entire Commonwealth of Pennsylvania can be found at the following link: [LiDAR Data](#).

2.2.5 Published Geological Information

Valuable information with respect to soils and geology is available through the Pennsylvania Department of Conservation and Natural Resources’ Bureau of Topographic and Geological Survey (DCNR and PGS), and much of the information is available electronically at the following link: [DCNR PGS Publication Index](#). The following publications and mapping products must be reviewed for all projects if applicable.

2.2.5.1 Pennsylvania Geological Interactive Maps

The Pennsylvania Geological Survey Interactive Map provides bedrock geologic mapping statewide. The user can locate the project location and determine what geologic formation(s) are present in and around the project. A brief description of the formation(s) is also provided. In addition to bedrock geology, earthquake epicenters, mapped sinkholes/closed depressions, and glacial limits are included. All the data presented on PGS interactive geologic map can be extracted to GIS or AutoCAD formats or to Google Earth. Customized PDF files of maps can also be created using the application/tool available on the website. The interactive geologic map is available online at the following link: [PA Geological Survey Interactive Map](#).

Similarly, valuable information can be found using eMapPA, which is a GIS based website and mapping tool that focusses on the display of geologically and environmentally relevant data. In addition to DEP-permitted facilities, there are over 50 map layers relating to administrative and political boundaries, culture and demographics, mining, streams and water resources, and transportation networks. This mapping website can be accessed at the following link: [eMapPA](#)

2.2.5.2 Pennsylvania Geological Publications

The PGS maintains a web page which includes links to geologic publications specific to Pennsylvania. The web page is located at the following link: [PGS PA Publications](#). This web page should be used to determine if relevant publications are available for the project area. If available, publications covering the project area should be reviewed for pertinent information. If a publication of interest is not available online, the PGS can be contacted through a link on the web page to determine how to obtain it.

PGS Map 61, *Atlas of Preliminary Geologic Quadrangle Maps of Pennsylvania*, compiled and edited by Berg and Dodge (1981) is no longer in print but can be accessed online at the following link: [PGS Map 61](#). This publication can be used to determine the geologic formation(s) underlying most project sites throughout the state. If the 7.5-minute quadrangle map which contains the project area is not included in Map 61, then the 7.5-minute quadrangle is addressed in greater geologic detail in another PGS publication. Note Map 61 is dated, and numerous geologic maps, publications, and GIS based online mapping applications have since been released providing more detailed geologic mapping than in Map 61.

PGS Environmental Geology Report 1 (EG-1), *Engineering Characteristics of the Rocks of Pennsylvania*, by Geyer and Wilshusen (1982) is also a very useful publication. After determining the geologic formation(s) present at the project site, EG-1 can be used to obtain preliminary engineering information about the project bedrock formation(s). Bedrock information in this publication includes: description of rock type(s); bedding, fracturing and weathering; drainage; foundation and cut slope stability; ease of excavating and drilling; and porosity and permeability. This publication can be accessed online at the following link: [PGS Environmental Geology Report 1](#). Bedrock geology information can also be obtained online from the PGS Interactive Geologic Map.

2.2.5.3 PGS Open-File Reports

The PGS completed a series of investigations to map karst features in carbonate rocks in Pennsylvania. Locations of surface depressions, sinkholes, and surface mines and caves were compiled. Results of these investigations were released in a series of county based open-file reports. The reports consist of a series of 7.5-minute quadrangle maps which contain the location of karst features on a geologic base map. A list of these open file reports can be found on the web page at the following link: [PGS Open-File Reports on Karst Features](#). Alternatively, the PGS maintains a sinkhole inventory online database at the following link: [PGS Sinkhole Inventory Database](#).

2.2.5.4 Open-File Miscellaneous Investigation (OFMI) Report 05-01.1

Open-File Miscellaneous Investigation (OFMI) Report 05-01.1, *Geologic Units Containing Potentially Significant Acid-producing Sulfide Minerals* (2005) includes a map of the state indicating areas where bedrock may contain sulfide deposits, which when exposed, could lead to the production of acidic drainage (i.e., acid bearing rock). This report should be consulted for all projects to determine if acid drainage may be a concern on the project so that appropriate planning/investigations can be taken. This publication is available online at the following link: [Open-File Miscellaneous Investigation Report](#)

2.2.5.5 Open-File Surficial Geology (OFSM) Reports

Surficial geologic maps depict unconsolidated deposits overlying bedrock. These maps are primarily concentrated in the north and northeast glaciated portions of the state and provide detailed landform information. These open-file reports are available online at the following link: [Open-File Surficial Geology Reports](#)

If the project area is covered by one these reports, they are very useful as they map surficial deposits that are prone to slope stability problems and identify surficial deposits that are known to cause slope stability issues. The maps also contain isochore lines which indicate the approximate thickness of surficial overburden deposits. This information can be used to preliminarily determine approximate boring depths, which is an important aspect of planning a subsurface exploration program. The surficial geology maps portray a variety of deposits, including: alluvium, boulder deposits, alluvial fans, wetlands, fill deposits, colluvium, glacial till, glacial lake clay deposits, and bedrock.

2.2.5.6 Landslide Susceptibility

The USGS has published numerous Professional Papers, Open-File Reports, and Miscellaneous Field Studies Maps related to landslide susceptibility. These publications include mapping of landslide susceptible areas and inventory of historical and active landslides. The majority of these reports and maps focus on the Pittsburgh and surrounding region. These publications are available from the USGS Publications Warehouse at the following link: [USGS Publications Warehouse](#). Initiate a search for “Pennsylvania Landslides” to get a list of available publications.

If the project is within the Williamsport 1- by 2-degree quadrangle, the PGS Environmental Geology Report titled *Landslide Susceptibility in the Williamsport 1- by 2-degree Quadrangle, Pennsylvania*, by Delano and Wilshusen (1999) must be reviewed. This publication includes a landslide susceptibility map and supporting text. This publication is not available online; however, it can be purchased from the PGS at the following link: [Landslide Susceptibility in the Williamsport Quadrangle](#).

2.2.5.7 Stratigraphic Columns

The Department of Conservation and National Resources (DCNR) and the Pittsburgh Geological Society provides stratigraphic columns for central and western Pennsylvania to represent the vertical location of rock units. A typical stratigraphic column displays a sequence of rocks arranged with the youngest rock unit at the top and the oldest rock unit at the bottom. The majority of these figures focus on the Pittsburgh and surrounding region. The DCNR stratigraphic column figures are available from the following link: [DCNR Figures](#). The Pittsburgh Geological Society stratigraphic column of the Pittsburgh region is available from the following link: [Pittsburgh Figure](#).

2.2.6 Other Maps and Publications

2.2.6.1 Coal Deposits and Mine Maps

Subsidence associated with the underground extraction of coal is a prevalent geologic hazard. After coal is removed from the ground, the roof of the mine begins to collapse. The sagging of rock layers may propagate to the surface. Land subsidence can be devastating to a roadway, bridge foundation, retaining wall, or other highway structure. According to the Pittsburgh Geological Society, areas situated more than 200 feet above an abandoned room and pillar mine generally do not suffer major damage from subsidence caused by abandoned mines.

Coal deposits and mined coal seams are predominantly located in the Main Bituminous Field in the western part of Pennsylvania. There is a small bituminous field in the north central part of the state (primarily District 3-0), and there are some anthracite coal fields in the eastern (central and northern) part of the state. These coal fields are shown in the PA DCNR Map 11 available at the following link: [Distribution of Pennsylvania Coals](#)

If a project site is located within the Bituminous or Anthracite Coal Fields of Pennsylvania, research should be performed to determine if the project site is subject to mine subsidence. The Pennsylvania Department of Environmental Protection (DEP), Bureau of District Mining Operations should be contacted to obtain a Coal Status Report for the project site. Currently, the California District Mining Office handles all requests for obtaining mining information at a project site. Information on how to obtain a Coal Status Report can be found at the following link: [Coal Status Report](#).

The PGS has numerous publications related to coal deposits and mining. Mineral Resource Reports M 89 thru M 94 are useful because they contain information on coal crop lines, mined-out-areas and structure contours for several counties, including: Allegheny, Butler,

Fayette, Clarion, Washington, and Westmoreland. These publications can be found at the following link: [Mineral Resource Reports M 89 thru M 94](#).

The Pennsylvania Mine Map Atlas is also a useful source for coal mining and oil/gas well information. This Atlas contains Works Progress Administration (WPA) Mapping that was drawn in the 1930's. All mining shown on these maps is assumed to have taken place prior to 1935, and "active" oil/gas wells shown on these maps most likely are no longer active. This Atlas can be found online at the following link: [Mine Maps](#).

Historical mine maps can be obtained from the National Mine Map Repository (NMMR) online at the following link: [NMMR Historic Mine Maps](#). The actual maps are not available online. The website only lists a map inventory to determine which maps are available. To obtain actual copies of mine maps, contact the NMMR directly. Contact information is available on their website. There is a processing fee charged by NMMR for mine map requests. Small requests are typically sent via email, but for larger requests the mine maps are copied to a CD and sent by mail. The entire mine map collection of the NMMR resides in the Pittsburgh, PA office.

2.2.6.2 Flood Maps

Flooding on a project site can be destructive and life-threatening, so it is important to account for the impact of possible flooding from rising river levels or more extreme precipitation. Land-borne flooding may occur in rivers when the flow rate exceeds the capacity of the river channel and overflows banks or may occur due to an accumulation of rainwater on saturated ground. Planning for flood safety involves many aspects of analysis and engineering that should be considered during office reconnaissance, including:

- Observation of previous and present flood heights and inundated areas
- Statistical, hydrologic, and hydraulic model analyses
- Mapping inundated areas and flood heights for future flood scenarios
- Long-term land use planning and regulation

Each topic presents distinct yet related questions with varying scope and scale in time, space, and the people involved. USGS provides useful flood resources and hydraulic maps (historic and interactive) that includes water availability, flood areas, surface drainage precipitation and climate, geology, availability of ground and surface water, water quality and use, and streamflow characteristics. These maps are available from USGS at the following link: [USGS Flood Resources](#). Federal Emergency Management Agency (FEMA) has Flood Maps searchable by an address or longitude and latitude coordinates and are available at the following link: [FEMA Flood Maps](#). Also, Pennsylvania has a source for flood map information at the following link: [PA Flood Maps](#).

2.2.7 Water Well Inventory and Water Resource Publications

Often planning of a subsurface exploration involves drilling in proximity to water supply wells and springs. The construction phase of the project may involve blasting and/or large

quantities of earth moving. All of these activities can have the potential to adversely impact water supply wells in both quality and quantity. Therefore, it is necessary to research and locate any existing water supply wells or springs that may be adversely impacted. The following resource should be used to locate water supply wells and springs that may be impacted by a subsurface exploration or construction activities. In addition, the hydrologic information can provide some indication of the presence and depth of groundwater in terms of its impact on construction and influence on the engineering properties of soil and rock.

The PGS maintains the Pennsylvania Ground Water Information System (PaGWIS) which consists of a large Access database containing data for wells, springs, and groundwater quality throughout Pennsylvania. These records can be viewed online at the following link: [PaGWIS Groundwater Database](#).

2.2.8 Soil Survey Publications

General soils information may be obtained by consulting the U.S. Department of Agriculture (USDA) county soil survey report. These reports, which may be accessed through the local Natural Resources and Conservation Service (NRCS) office or online via the NRCS Web Soil Survey (located at the following link: [NRCS Web Soil Survey](#)), provide soils mapping overlain on aerial photographs and descriptions of soil map units. The soil survey provides tables of soil physical and chemical properties and suitability for various uses. Soil classifications in these publications are based on the USDA system, not the engineering classification systems (i.e., USCS or AASHTO) typically used for roadway/structure projects and are generally intended more for agricultural use. However, if site specific soil information is not available from test borings, these USDA publications can provide an indication of the soil conditions to be expected on a project site.

2.2.9 Previous Subsurface Exploration Data

If a project is on or near an existing alignment, previous subsurface information may be available and requested from the DGE/PGM. Existing geotechnical information and well drilling logs, if available, can contain relevant information. This information can be useful in setting preliminary boring locations and depths. The Department's gINT database will be a primary source to obtain previous subsurface exploration data. This database will be populated with gINT project files, and queries can be performed to search for nearby projects. At this time, only Central Office and Districts have access to this gINT database. Therefore, Districts can provide relevant gINT project files to Business Partners, or Business Partners can request that Districts provide any existing relevant gINT project files for reference.

Most Department projects consist of improvement or replacement of existing alignments, structures, and facilities. Often roadway geotechnical engineering reports, structure foundation reports, construction records, as-built drawings, and/or pile driving records for existing structures are available. If available, maintenance records for nearby roadways and structures can provide indications to subsurface conditions and provide long term characteristics of the site.

2.2.10 Previous Site Use

It is important to obtain and review any documentation of previous site use particularly those that could identify the potential for hazardous waste. The knowledge and identification of hazardous subsurface materials could affect the subsurface exploration approach as well as specific project needs. If potentially hazardous materials are discovered, adjustments to the subsurface exploration may need to be made to protect the onsite workers and to comply with environmental regulations. If during office reconnaissance or the subsurface exploration potentially hazardous substances are determined to be present within the project area, the PGM should notify the DGE and discuss potential adjustments to the subsurface exploration or the need for necessary precautions during subsurface explorations.

Other previous site uses to be considered is whether there is a potential for archeological artifacts to be discovered at the site or if the site was previously used as a drainage area or basin. If there is a potential for artifacts to be discovered, an archeological investigation should be completed prior to commencement of the subsurface exploration. The PGM should confirm with the DGE as to whether an archeological investigation is required prior to the commencement of the subsurface exploration. If the site was previously used as a drainage area or basin, the potential of an active sinkhole area should be considered.

2.3 FIELD RECONNAISSANCE

After review of the proposed alignments and available geotechnical information, field reconnaissance of the proposed alignments, including structure locations, is required. Ideally, perform field reconnaissance after the alignments have been field located with flagging/stakes by others. Field reconnaissance of all alignments is required, and the full length of the alignments must be viewed. However, concentrate on portions of the alignments identified as potentially significant (from a geotechnical perspective) based upon review of available geotechnical geological information and the proposed construction. Some examples include:

- Aerial photographs showing lush vegetation may indicate seeps and springs which could result in early warning signs for possible locations for landslides
- Mapped sinkholes, landslides, mines, spoil piles, etc.
- Areas of gently rolling terrain which are often favorable, but may also be an indicator of subsurface conditions unable to support steep slopes
- Rapidly changing contours and bodies of water which may indicate rapidly changing subsurface conditions

If significant rock cuts are proposed along an alignment, obtain representative rock discontinuity (bedding and joints) measurements from outcrops or cut slopes and identify potential needs for rockfall control and/or mitigation. These measurements can be used in a conceptual design stereonet analysis to estimate permissible rock cut slopes. Estimating slopes of deep rock cuts are valuable in determining approximate required limits of disturbance and earthwork balance.

Locate pertinent geotechnical features observed during field reconnaissance on project mapping. A camera and a hand-held GPS unit are very useful for performing this task, and a high level of accuracy is not necessary at this phase of the project. If necessary, flag areas of interest for later conventional survey by a licensed surveyor to locate features on project mapping. **Reconnaissance performed during the Alternatives Analysis Phase is intended to identify significant features that may have an influence on the selection of the preferred alignment, may require specific attention during later design phases, and will assist in scoping for the next phase of design.** More detailed field reconnaissance is conducted during the Preliminary Design Phase once the preferred alignment has been selected.

It is essential for the PGM to complete field reconnaissance to develop first hand appreciation of the geologic and topographic conditions, and become well-informed of geotechnical issues, site access, and drilling/construction conditions present at a project site. As previously stated, field reconnaissance should proceed only after completion of office reconnaissance and, if private property is involved, after Notice of Intent to Enter Letter(s) have been received by property owner(s). Completion of office reconnaissance aids in identifying geotechnical issues of concern that the PGM should focus on during field reconnaissance. All the information collected during office reconnaissance should be present during field reconnaissance. This enables the PGM to verify findings and document any site conditions that do not agree with the office component of reconnaissance including project maps, plans, sections, subsurface profiles, and as-builts, etc.

The main objective of field reconnaissance is to determine the key geotechnical issues that will have an influence on the project design. Some examples of field conditions that are important to geotechnical exploration and design include:

- Terrain
- Soil exposure
- Creek/Stream/River Beds/Banks/Scour
- Rock Outcrops
- Utilities
- MPT planning for drilling
- Wetlands
- Guiderail – post spacing
- Hazardous materials

2.3.1 Drilling and Boring Location Observations

During field reconnaissance, proposed test boring or geotechnical testing site locations and field conditions should be identified and/or reviewed, documented, and photographed. Field confirmation of these locations is required. The PGM must verify that the proposed boring location and drilling method will successfully obtain the required geotechnical information needed to complete the design. Correct placement of borings reduces or eliminates the need for additional borings which increase cost and time necessary to complete the subsurface exploration. During the review, document all potential and/or observed utilities, site restrictions,

required traffic control, private properties, environmental concerns, accessibility, and any other features or conditions that may impact the execution of a subsurface exploration.

Identify the type of drilling required or confirm that the proposed type of drilling is actually the best suited for the given site conditions. Revise drilling methods according to site conditions if necessary. Note potential drilling issues such as shallow groundwater, artesian conditions, loose or heaving sands, cobbles or boulders, pinnacled rock, and voids. Local experience and knowledge (including local residents), topographic maps, and well information are a few of the more valuable tools to identify potential artesian conditions. Every effort should be made to identify potential artesian conditions as these could be especially problematic during drilling operations. If the investigation proposes special sampling equipment, verify its applicability and practicality during field reconnaissance.

Complete an initial determination of what type of drilling equipment is best suited for the terrain and the anticipated subsurface conditions. Depending on the project, more than one type of drill rig may be needed to complete the subsurface explorations. Exploration equipment, drill rigs, grout mixers, and support equipment access should be reviewed and photographed to determine if site access improvement will be required prior to commencement of the subsurface exploration program. If access improvement will be required, document appropriate equipment needed such as a bulldozer or skid-steer. In addition, if water will be required for drilling, a source of water should be identified and documented along with any conveyance needs to supply water to the drilling equipment.

Observe and document any potential problems with utilities such as overhead and underground power lines, gas lines, and telecommunications lines. Although utility clearances (i.e., PA One Call) will need to be obtained before the subsurface exploration begins, the locations of the utilities can influence where borings can be located and ultimately the design of the project.

Observe and note traffic control requirements per Publication 213 to accomplish the field exploration program. Consider the practicality of the proposed subsurface exploration program with respect to public inconvenience and travel delays.

If borings are planned over water, reconnaissance should note the required size of barge best suited for the job, locations for launching the barge, depth of water, and barge anchoring details.

2.3.2 Environmental/Property Damage Considerations

Identify and document any potential impacts the exploration program may have on local groundwater and surface water. Note the presence of any wetlands. If the project is near a watercourse determine if erosion and sedimentation control is required to prevent or mitigate the amount of turbid drill water from entering the watercourse. Determine if property damage will or may result from the movement of drilling equipment and drilling activities. For example, trees may need to be cut or benching may need to be completed. Anticipate what types of site restoration will be required to repair potential site and property damage.

2.3.3 Site Conditions

There are numerous features and observations that are important with respect to the geotechnical aspects of the project. Some of these features that should be considered while performing field reconnaissance are: condition/performance of existing facilities (pavements, embankments, cut slopes, guiderail, structures, etc.), soil and rock exposures, subsidence/sinkholes, mine subsidence, acid drainage, dead/stressed vegetation, stained ground, scarps, seeps, wetlands, hummocky terrain or bulges, bent or tilted trees, tension cracks in the ground or pavement, tilted or irregular guiderail, and evidence of acid bearing rock.

2.3.3.1 Rock Exposures/Outcrops

Examine and/or photograph outcrops or exposures that warrant future investigation (detailed structural/discontinuity mapping). Preliminarily assess the stability of existing rock slopes and evaluate relative performance. Measure orientation of bedding and cut slope angle. If possible, indicate rock folding and faulting, seepage, joint patterns, and strata breaks or marker beds.

2.3.3.2 Sinkholes/Karst

For project areas underlain by carbonate bedrock (limestone and dolomite), the PGM should confirm the presence of mapped or known sinkholes and closed depressions, identify any other karst features not currently mapped, and determine if there is a known sinkhole history. Determine if surface mining operations are located nearby. If karst features are identified, their dimensions should be measured and photographed if it is safe to do so.

2.3.3.3 Mine Subsidence

For projects located in the anthracite or bituminous coal fields of Pennsylvania, subsidence associated with underground mining is of concern. After coal is removed from the ground, the roof of the mine often begins to collapse over time. The sagging of rock layers may propagate to the surface resulting in land subsidence. The PGM should cautiously and thoroughly investigate the project site for signs of mine subsidence, especially if the review of mine maps indicate mining has occurred less than 200 feet beneath the project.

2.3.3.4 Seeps/Perched Groundwater

Seeps are probably the most common feature present in landslides and landslide prone areas. Seeps typically indicate high or perched groundwater. High groundwater causes soil to become saturated, which in turn decreases the shear strength of the soil. Additionally, saturated soils are heavier than unsaturated soils and therefore cause increased driving force, which promotes slope instability. Saturated soil can also result from broken or leaky water (potable, storm drainage, or sewer) pipes and from concentrated flows of stormwater runoff. Often seeps and wet areas will be characterized by areas of lush vegetation and vegetation typical of wetlands.

2.3.3.5 Unstable Slopes/Landslide Features

Hummocky terrain is typically present in areas of current or past landslides or unstable slope activity. Slope failures generally do not produce uniform displacement of soil throughout the sliding mass. Additionally, larger slope failures often have smaller failures develop within the larger slide area. Thus, where landsliding has occurred, the ground surface is generally a hummocky or non-uniform slope.

Where trees are present on a slope that is moving, or has moved, the trees will typically be tilted or bent. If both new and old growth trees are tilted, that is an indication that failures are more recent. If only old growth trees are tilted and bent and new growth trees are straight, then slope movement most likely occurred in the past and the slope is currently not moving. The limits of the tilted/bent trees can be a good indication of the limits of the landslide.

More obvious signs of slope failures include the presence of scarps, tension cracks in soil and/or pavement, or tilted/misaligned guiderail. Scarps develop at the failure surface, where material has moved downward, usually at the top and along the sides of the slide. Scarps, particularly associated with active landslides, are commonly sparsely vegetated. When a failure occurs, earth materials move downhill, and the materials at the top of the slide experience tension. If enough movement occurs, cracks may form perpendicular to the direction of movement at the crown of the landslide. These cracks, sometimes referred to as tension cracks, are typically continuous or are made up of numerous smaller cracks and generally extend the entire width of the landslide.

If a slide is active, these tension cracks will typically widen as movement continues. Tension cracks may not be present in areas where landsliding occurred in the past and is not ongoing due to erosion and filling of the tension cracks. Tension cracks can form in pavement, which are oftentimes more pronounced due to the rigid nature of pavement compared to soil.

Where roadways with guiderail exist within a project, the alignment of the guiderail can provide an indication of slope movement. Guiderail that is tilted or is not in a relatively consistent alignment most likely indicates slope movement. This movement may be more of a surficial nature, particularly where guiderail is installed near the top of the slope and the slope is steeper than 2H:1V. Misaligned guiderail can also result from poor compaction near the edge of the slope.

2.3.3.6 Structures

Prior to field reconnaissance for structure replacement or widening projects, review past inspection reports. During field reconnaissance for a subsurface exploration in support of a structure design, document and photograph any existing structures or abandoned foundations that may impact the design and construction of the new structure. Look for any signs of settlement or lateral movement. If suspected, surveying may be required to determine the magnitude of movement.

For structures which span bodies of water, inspect structure footings if exposed and the stream bank up and downstream from the structure for evidence of scour. The presence of rip-rap around a bridge foundation may indicate a past scour problem, particularly for older structures. Movement of rip-rap can also be an indication of scour. Observe and document if cobble or boulder size material is present within the stream bed because this can impact drilling production and/or drilling technique.

2.3.3.7 Hazardous Materials

Discuss potential for encountering hazardous materials on the project site with project environmental staff. It is likely that a Phase I Environmental Site Assessment has been completed prior to field reconnaissance. If documentation indicates the project site does contain or may contain hazardous materials, special attention should be made during field reconnaissance to locate and further document these areas. Some signs of the presence of hazardous materials include: prior land use (e.g., gas stations), stained/mottled soil, little or no vegetation, and odors. In addition, observe and document any signs of previous drilling investigations or excavations such as patched or damaged asphalt, and hummocky ground surface. Observe the topography of the project site and determine the general groundwater flow direction. Look for signs of active or historic remedial activities such as monitoring wells, and onsite remediation systems (e.g., vent pipes). Information pertaining to the procedure if hazardous/contaminated materials are encountered during any phase of the geotechnical investigation is specified in Publication 222, Section 103.09(e).

2.3.3.8 Acid Bearing Rock

The presence of acid bearing rock (ABR) is widespread in Pennsylvania and could have a significant impact on a project; therefore, it is important to determine if it may be present as early as possible. As discussed in [Section 2.2.5.4](#), the Pennsylvania Geological Survey has produced a map of potentially acid bearing rocks (OFMI Report 05-01.1). This reference should be the first step in determining if a project may be impacted by ABR. Check this map and any other publications that may be available to determine if the project lies in an area where ABR is known to be present. Also, check if past local projects encountered ABR. During field reconnaissance, observe and document any indications of the presence of ABR, such as yellow or orange precipitate in stream bottoms and/or heavily corroded metal pipes. As indicated in OFMI Report 05-01.1, in most cases in Pennsylvania acidic drainage involves the weathering (i.e., exposure to air and water) of iron sulfide minerals. The most common iron sulfide mineral is pyrite, and the

less common are pyrrhotite and marcasite. Coal, dark/black shales, rocks with pyritic minerals, and rocks with low amounts of calcareous minerals (i.e., acid buffering minerals) should be especially considered as having the potential to produce acidic drainage. If any indications of ABR are documented or observed on a project, follow the provisions presented in [Chapter 10](#), Acid Bearing Rock, for additional information.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

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GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 3 – SUBSURFACE EXPLORATIONS

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3.1 INTRODUCTION

This chapter of this publication provides guidelines, recommendations, and considerations for planning and executing subsurface exploration programs. Included are minimum requirements for number, location, and depth of test borings depending upon the complexity of the planned construction, the complexity and severity of the geologic conditions, and the risks and costs associated with system failure and/or service interruption. This chapter also discusses drilling equipment, tools, procedures, and other topics associated with subsurface explorations not addressed in Publication 222.

The subsurface exploration program is one of the most important geotechnical tasks for the majority of projects. A properly planned and executed exploration program will provide the information needed to perform geotechnical analyses and develop geotechnical design recommendations. Consider that all transportation facilities and related assets are constructed on earth materials (e.g., soil and rock). Next, consider that unlike virtually all other construction materials (e.g., concrete, steel, aggregates, etc.), earth materials are not manufactured so there are no manufacturing quality control programs for these unprocessed natural earth materials. Lastly, realize that there is an extremely high level of variability in these earth materials. Consequently, the subsurface exploration and laboratory testing programs are the main tools used to estimate the earth material properties/characteristics that underlie a site.

It must be understood that all projects are unique, and the subsurface exploration program for each project must be carefully contemplated and planned to fit the needs and constraints specifically associated with the project. **Simply using the minimum requirements indicated in this chapter for preparing the subsurface exploration program is not an acceptable approach – they are absolute minimum requirements.** While there may be instances of relatively uniform, non-complex, favorable subsurface conditions where these minimum requirements could suffice, following the absolute minimum requirements will often result in inadequate or insufficient subsurface information to efficiently (i.e., safely and cost effectively) design the geotechnical features of the project.

Prior to planning the subsurface exploration, several tasks discussed in [Chapter 1](#) of this publication must be completed. These include: review of plans, profiles and cross-sections; review of available geotechnical data; and field reconnaissance. As previously discussed, the proposed subsurface exploration program must be documented in a “Subsurface Exploration Planning Submission”. Requirements for this submission are included in [Chapter 1](#) of this publication. This submission must be reviewed and approved by the District Geotechnical Engineer (DGE) prior to performing the exploration. The subsurface exploration must be administered and performed in accordance with all requirements indicated in Publication 222.

A useful reference when planning or performing a subsurface exploration program is the latest version of “Subsurface Investigations – Geotechnical Site Characterization”, Publication No. FHWA NHI-01-031.

3.1.1 Purpose

The purpose of a subsurface exploration is to characterize the materials (soil, rock, and groundwater) underlying the project site. Characterization includes identifying the material types, physical properties, depths, thicknesses, and ground water conditions present at the specific boring/testing location. This includes collecting soil and rock samples for laboratory testing in order to establish the soil and rock parameters needed for geotechnical analyses and design. The detail to which these materials must be characterized will depend on several factors, including: the design phase of the project (i.e., Alternatives Analysis, Preliminary or Final Design), the complexity of the proposed construction, the complexity of the subsurface conditions, and the risks and costs associated with failure of the various facilities and sub-facilities involved, and service interruption. As examples:

- A subsurface exploration performed during the Alternatives Analysis Phase will be less comprehensive than explorations performed during the Preliminary and Final Design Phases.
- Complex projects, like new alignments and large structures, and projects in urban settings, will require a more detailed exploration compared to less complex projects, like minor roadway or bridge widenings and bridge replacements.
- Projects underlain by complex subsurface conditions, like carbonate rock, folded and faulted stratigraphy, mined areas, problematic soils, landslide prone areas, etc., will require a more detailed exploration compared to less complex subsurface conditions like horizontal stratigraphy or dense/competent overburden soils.
- Projects involving limited access and/or high ADT routes, significant potential service interruption, routes for emergency services, major economic impacts, excessive detours, or other similar high risk or major consequence conditions may dictate a more extensive subsurface exploration to mitigate risks and/or severe consequences.

As a project progresses through the various design phases, information obtained from previous phases must be used to the fullest extent possible in the subsequent design phases. At a minimum, the previously obtained information should aid planning of future subsurface explorations and, where possible, supplement, complement, or verify final design explorations. When possible, borings drilled during the Alternatives Analysis and Preliminary Design Phases of a project should be located and drilled to depths that meet the requirements of the Final Design subsurface exploration. When possible and appropriate, in-situ testing should also be performed to aid in optimization and efficiency of Final Design.

3.1.2 Subsurface Characterization

The following must be performed to adequately characterize the subsurface conditions at a project site:

- Identify the subsurface strata (soil and rock), including physical description, stratum thickness, soil origin (e.g., residual, alluvial, colluvial, etc.), and stratum top and bottom elevations

- Collect information to assess engineering properties of soil strata, including: color, grain-size distribution, density/consistency, structure, cohesion, and moisture content
- Collect information to assess engineering properties of rock strata, including: rock type, color, hardness, weathering, rock quality designation, condition of joints, and presence of solutioning
- Record groundwater level(s), preferably obtaining seasonal variations
- Obtain soil and rock samples for laboratory testing

Other items that may have to be completed to characterize the subsurface include:

- Installation and monitoring of instrumentation (e.g., piezometers, inclinometers, etc.)
- Performing in-situ tests (e.g., cone penetrometer testing (CPT), vane shear testing (VST), pressuremeter testing (PMT), permeability, etc.)
- Conducting geophysical survey(s) (e.g., electrical resistivity, seismic refraction, etc.)

3.1.3 Subsurface Exploration Methods

There are several methods that can be used to obtain subsurface information, as follows:

- Test borings (e.g., standard penetration testing (SPT), thin-walled tube sampling, auger sampling, rock core drilling)
- In-situ testing (e.g., CPT, VST, PMT, etc.)
- Geophysical Investigations
- Test pits

Although all of these methods should be considered when planning a subsurface exploration, test borings are required to be performed on all projects where it is necessary to collect subsurface information and samples. Test borings alone are often adequate on small and/or less complex project sites, whereas large, complex projects with adverse geologic conditions may require the use of additional subsurface exploration methods. As needed and where beneficial, other subsurface exploration methods may be used to augment the test boring information. For example, test pits are a low cost, efficient method of subsurface exploration, for generally shallow conditions.

3.2 TEST BORINGS

Test borings are very versatile and are most commonly used in Pennsylvania for collecting subsurface information for transportation projects. They can be performed in most locations and are capable of being advanced through most subsurface conditions. Test borings are capable of meeting the main goals of a subsurface exploration including: identifying soil and rock strata, characterizing the strata, measuring groundwater levels, and collecting samples for laboratory testing. Test borings can be performed with a variety of drilling equipment, tools and methods. Methods for advancing test borings, and detailed requirements for recording

information/data obtained from test borings must be performed in accordance with Publication 222 and applicable AASHTO and ASTM standards referenced in Publication 222. Any subsurface exploration sampling, testing, monitoring or instrumentation not specifically addressed in Publication 222 or Publication 293 must follow relevant AASHTO or ASTM standards, unless indicated or specified otherwise.

3.2.1 Planning Subsurface Exploration Program

As a project advances through the various design phases (i.e., Alternatives Analysis, Preliminary Design and Final Design) the requirements of the subsurface exploration program become more comprehensive. During the Alternatives Analysis Phase, the subsurface exploration is intended primarily to characterize subsurface conditions sufficiently to assist in selection of a preferred alignment. As the design becomes more focused and detailed in the Preliminary and Final Design Phases, the subsurface exploration program also becomes more focused, comprehensive, and thorough in order to obtain subsurface information necessary for design purposes and to adequately identify and investigate any areas of concern or significance that may have an impact (positive or negative) on the project.

The complexity of the project, subsurface conditions, and risk factors form the basis for developing the subsurface exploration program. The higher the level of complexity or more significant the risk factors in either the project or the subsurface conditions, the greater and more focused the level of effort (e.g., number and depth of borings, instrumentation, etc.) is required during the subsurface exploration.

The availability of reliable existing subsurface information will help determine the scope of the proposed subsurface exploration program. Projects with available subsurface information that is poor in quality, not sufficient or verifiable at the time of planning, generally require more exploration (i.e., greater number and depth of borings and sampling) compared to projects where the subsurface conditions have some existing reliable characterization. In situations where apparent, reliable information is available, the exploration may be more focused and limited. In all cases, however, sufficient exploration must be conducted to verify any subsurface information not obtained by the final designer (i.e., Department or consultant) for design-bid-build projects. For design-build projects, adequate and reliable subsurface information must be provided as part of the geotechnical design guidance report to permit potential project teams to produce a design concept that can be bid with a level of risk consistent with a comparable design-bid-build project. Adequate is defined as a comparable level of subsurface exploration necessary to permit a design consultant to produce a design to the standards and quality expected for a normal design-bid-build project.

For large projects, it is frequently beneficial to have at least two subsurface exploration programs or phases. The first exploration program, during the Alternatives Analysis or Preliminary Design Phase, is intended to gain a broad understanding of the subsurface conditions. Additional exploration, during the Preliminary and/or Final Design Phases, can focus on the areas of the project where complex construction is proposed and where complex subsurface conditions that impact proposed construction are anticipated. A single subsurface exploration program will most likely be sufficient for small projects.

3.2.2 Subsurface Exploration Planning Meeting

A meeting between the Project Geotechnical Manager (PGM) and DGE should be considered prior to finalizing the subsurface exploration program. This meeting is strongly encouraged for large and/or complex projects and for projects with complex geology and is especially important for large/complex projects where a field meeting between the PGM and DGE was not previously held. The need for this meeting should be identified during development of the scope of work for the project design phase.

There are several approaches when conducting a subsurface exploration planning meeting. For large projects, it is generally necessary to conduct an office meeting to review the proposed construction in context with the proposed subsurface exploration program. The office meeting would be followed immediately (i.e., the same or next day) by a field view meeting to review the proposed exploration plan in context to both existing and proposed field conditions. On small projects, either an office meeting or a field meeting may be sufficient. A field meeting should be conducted if the DGE has not previously viewed the project site. The subsurface exploration planning meeting should be conducted after the search of available/published literature and field reconnaissance has been completed and after the proposed subsurface exploration program has been developed by the PGM. After the meeting, the subsurface exploration program must be finalized and submitted to the DGE for review and approval. The Test Boring Plan and Schedule of Proposed Borings must be included in the “Subsurface Exploration and Planning Submission”. Refer to [Chapter 1](#) of this publication for the requirements of this submission. A generalized laboratory testing program must also be submitted with the proposed subsurface exploration plan for approval. This laboratory testing program will be finalized after the subsurface exploration is completed.

3.2.3 Test Boring Plan and Schedule of Proposed Borings

The main products/deliverables of the “Subsurface Exploration and Planning Submission” are the Test Boring Plan, Schedule of Proposed Borings, and the generalized or preliminary laboratory testing program. The Test Boring Plan shows the location of the test borings. The Schedule of Proposed Borings, which is included in Attachment 1 of the Subsurface Boring, Sampling, and Testing Contract (SBSTC), provides estimated test boring depths, boring location (station, offset, and/or coordinates), soil sampling type(s) and intervals, rock core drilling type and length, and other requirements as needed (e.g., instrumentation, grouting, MPT, etc.) for each individual test boring. The Test Boring Plan and Schedule of Proposed Borings, once approved by the DGE, become part of the SBSTC. Any proposed changes to the plan or schedule after the initial approval must be submitted to and approved by the DGE.

Development of the Test Boring Plan and Schedule of Proposed Borings is dependent upon a variety of factors including: the design phase of the project, the complexity of the proposed project, the complexity of the subsurface conditions, and the availability and reliability of existing subsurface information. Although there are similarities and generalities for any project when planning a subsurface exploration, each project is unique with its own set of

knowns, unknowns, constraints, complexity, geologic setting, etc. and must be planned per the needs of the specific project.

3.2.4 Guidelines for Establishing Number, Location, and Depth of Test Borings

Development of the subsurface exploration program, including the number, location, and depth of test borings, must account for a range of both general and project specific conditions. As discussed above, all projects are unique, and specific requirements that address every situation with respect to number, location and depth of test borings relative to project needs and conditions, cannot be provided in this publication. Instead, guidelines are provided in this publication to aid in assembling an adequate and efficient subsurface exploration plan.

The Test Boring Plan and Schedule of Proposed Borings are developed based on the best available information. The best available information often includes published information, field reconnaissance, and the proposed construction. Site specific subsurface information, such as previously drilled borings, may or may not be available. If available, consideration must also be given to the accuracy, reliability, and applicability of the information. The actual conditions encountered may differ significantly from those indicated from existing available information. It is absolutely essential that an adequate number of confirmatory borings be conducted to verify the accuracy of any historical subsurface information. Unless existing subsurface information was obtained from a previous phase of the current project or was obtained within the last 10 years using current subsurface investigation standards and practices, never rely solely upon historical subsurface data.

It is critical that the PGM maintain close contact with the drilling inspector(s) during the subsurface exploration to evaluate the actual subsurface conditions and the applicability, accuracy, and reliability of existing information, and to make modifications to the program as necessary to obtain sufficient reliable information to permit preparation of a final design. Close communication helps ensure the necessary information is obtained and helps prevent excessive/costly drilling. Any changes to the approved subsurface exploration program must be approved by the DGE. It is also absolutely critical that the certified drilling inspector keep thorough and accurate records of all drilling operations with thorough daily documentation of progress, directives, compliance, safety, traffic control, and any significant or unusual events or encountered conditions.

Tables [3.2.4-1](#) and [3.2.4-2](#) provide requirements for the minimum level of effort for subsurface exploration. It is not possible to provide a single comprehensive set of requirements or recommendations that address the needs of every project, nor is it appropriate to do so. The Department obtains the services of qualified professionals to assess the specific needs and conditions of individual projects and the unique subsurface conditions and challenges that every project presents. Therefore, there is an expectation that sound professional technical assessment be used to determine, plan, and execute the level of effort necessary to adequately identify and define the project specific subsurface conditions to a degree consistent with the project complexity and risk.

Table 3.2.4-1 – Number and Location of Test Borings

Area of Exploration	Design Phase ¹	Number and Location of Borings ² (Listed herein are absolute minimums that do not consider complexity of geologic conditions, project or site complexity, or risk factors.)
Roadway ³	Alternatives Analysis	Approximately one boring every 1,000 feet or more. Consider placing at least one boring in each section of proposed deep cut and fill. Where topography varies considerably in proposed deep cut/fill, also consider two transverse borings to develop subsurface cross-section.
	Preliminary Design	Approximately one boring every 500 to 1,000 feet. Place at least one boring in each section of proposed deep cut and fill. Where topography varies considerably in proposed deep cut/fill, provide two transverse borings to develop subsurface cross-section.
	Final Design	Approximately one boring every 300 feet or less. Where topography varies considerably in proposed deep cuts/fills, and at critical analyses locations (e.g., slope stability, settlement, etc.), provide two or more transverse borings to develop subsurface cross-sections.
Bridge ^{3,4}	Alternatives Analysis	One boring per structure. Consider an additional boring for long bridges where topography and geology varies considerably.
	Preliminary Design	Same as above. If borings were drilled during the Alternatives Analysis Phase, additional borings may not be necessary for the Preliminary Design Phase.
	Final Design	Provide a minimum of two borings per substructure unit. Provide a minimum of three for substructures 100 feet or more in length/width. Provide an additional boring for each wingwall over 25 feet in length. For wingwalls over 50 feet long, follow guidelines for Retaining Walls. For bridges supported on non-redundant, large diameter drilled shafts, provide one boring located within the footprint of the shaft. One boring per substructure unit may be acceptable for small (less than 50 foot span), two lanes, low ADT bridges underlain by flat geology if the borings indicate uniform subsurface conditions.
Culvert ^{3,4}	Alternatives Analysis	One boring per culvert.
	Preliminary Design	See guidelines for Bridge above.
	Final Design	Provide minimum of two borings (one at inlet and one at outlet). Provide additional borings for culverts over 100 feet long (maximum boring spacing is 100 feet). Provide borings for each wingwall over 25 feet in length. For wingwalls over 50 feet long, follow guidelines for Retaining Walls.
Retaining Wall ^{3,4}	Alternatives Analysis	Minimum one boring per wall with a maximum spacing of 1,000 feet. Consider additional boring(s) if topography varies

Area of Exploration	Design Phase ¹	Number and Location of Borings ² (Listed herein are absolute minimums that do not consider complexity of geologic conditions, project or site complexity, or risk factors.)
		considerably along wall. Place within footprint of wall, or on face for linear walls.
	Preliminary Design	Minimum two borings per wall with a maximum spacing of 500 feet. Place within footprint of wall. For cut walls, consider placing additional borings just behind the active earth pressure wedge.
	Final Design	Minimum two borings per wall with a maximum spacing of 100 feet. Stagger borings between the front and back of the proposed footing. For cut walls, place additional borings with a maximum spacing of 300 feet just behind the active earth pressure wedge.
Sound Barrier Wall ^{3,4}	Alternatives Analysis	One boring per 1,000 feet of sound barrier wall. Consider additional boring(s) if topography varies considerably along wall.
	Preliminary Design	Minimum two borings per wall with a maximum spacing of 500 feet.
	Final Design	Minimum two borings per wall with a maximum spacing of 200 feet.
Pavement Design ⁵	Preliminary Design	Approximately one boring every 1,000 feet or more to collect sample for CBR testing. Borings are not needed in proposed roadway fill sections. Obtain pavement core(s) for match-in-kind and overlay projects. Consider using roadway boring to collect CBR sample.
	Final Design	Same as preliminary design except one boring every 500 to 1,000 feet.
Dynamic Message Sign	Final Design	One boring per foundation.
Special Conditions	Mined Areas	Projects located in areas where mining (deep or strip) has occurred will most likely require borings in addition to the guidelines discussed above for all design phases. Utilize available mine maps to help locate borings. Consider the use of additional exploration techniques like air-track drilling and geophysics. The subsurface exploration technique(s) must encompass each substructure unit and the entire structure site.
Special Conditions (cont.)	Carbonate Geology (Limestone, Dolomite, etc.) (cont.)	Projects located in areas of carbonate geology may require borings in addition to the guidelines discussed above for all design phases. However, it may be more beneficial and cost effective to use other additional exploration techniques like air-track drilling and geophysics where borings indicate extremely variable subsurface conditions. The subsurface exploration technique(s) must encompass each substructure unit and the entire structure site.

Area of Exploration	Design Phase ¹	Number and Location of Borings ² (Listed herein are absolute minimums that do not consider complexity of geologic conditions, project or site complexity, or risk factors.)
	Claystone/ Mudstone Geology	Projects located in areas of claystone or mudstone geology may require borings in addition to the guidelines discussed above for all design phases.
	Landslides	Cross-section borings through the landslide are required. Locate borings within the landslide near the head scarp and toe of the landslide. Add intermediate borings when the length (transverse) of the landslide exceeds approximately 200 feet. Place cross-section borings every 200 to 300 feet (longitudinally) along the slide. Also, provide borings above the landslide head scarp.
	Acid Bearing Rock	See Chapter 10 of Publication 293 for boring requirements.
	Temporary Support of Excavation	Consider adding boring(s) in area(s) of anticipated temporary shoring if structure borings are not sufficient with respect to location and/or depth.

- Notes: 1. Previously discussed in this publication, the goal of the subsurface exploration during the Alternatives Analysis Phase is to gain a general understanding of the subsurface conditions, and to identify favorable/unfavorable geotechnical conditions to aid in the selection of the preferred alignment. Also, on some projects only one subsurface exploration program may be necessary, and it may be conducted during the Preliminary Design Phase. If this is the case, the guidelines for the number and location of borings for the Final Design Phase must be followed.
2. In addition to the design phase, the number and location of borings will depend upon the geologic complexity of the site and the complexity of the proposed construction. In general, projects with complex geology (e.g., folded and/or faulted stratigraphy, carbonate geology, mined areas, etc.) will require more borings compared to projects situated in less complex geology (e.g., horizontal stratigraphy, etc.). Additionally, more borings will generally be required for projects with deep cuts, high fills, long span bridges, high retaining walls, etc.
3. Where possible utilize structure borings to also satisfy roadway boring requirements.
4. Ideally locate borings within the footprint of the proposed foundation, except as discussed with retaining cut walls. The adequacy of borings drilled outside of the foundation footprint and the acceptable distance outside of the foundation should be determined on a case by case basis. Borings drilled outside of the footprint for lightly loaded foundations (approximately 2 to 3 tsf.) and/or in non-complex, uniform geology may be acceptable if they are approximately ten feet close. Borings drilled outside of the footprint for heavily loaded foundations and/or in complex, folded and/or carbonate geology generally will not be acceptable.

Where borings cannot be drilled within the footprint of the proposed foundation, borings must enclose footprint.

5. See [Section 3.7.10](#) for further details about borings and sampling for pavement design purposes.

Table 3.2.4-2 – Depth of Test Borings

Area of Exploration	Boring Depth^{1,2,3}
Roadway	<p>Embankments – Extend borings a minimum of two times the proposed embankment height unless competent material (medium dense granular or hard cohesive) with sufficient thickness is encountered.</p> <p>Cuts – Extend borings ten feet below the proposed subgrade elevation.</p> <p>At-grade areas – Extend borings <u>minimum</u> of five feet below proposed subgrade. Borings may need to extend deeper depending upon known or anticipated adverse or complex geologic conditions, or other factors where increased boring depths may be necessary to adequately define subsurface conditions.</p>
Bridge	<p>Spread footing on bedrock, piles bearing on bedrock, and micropiles socketed into bedrock – For non-carbonate bedrock extend borings ten feet below estimated bottom of footing/pile tip elevation. If potentially slaking rock (claystone or mudstone) is encountered at bottom of boring, continue boring up to an additional ten feet (maximum) in an attempt to encounter more competent rock. For carbonate bedrock and areas where deep mining has occurred extend borings 20 feet below estimated bottom of footing/pile tip elevation. Note: Minimum pile length for integral abutments is 15 feet per DM-4, but for long spans additional length may be needed to satisfy bending. If bedrock is shallow, ensure that sufficient bedrock is cored to extend ten feet minimum below estimated pile tip elevation.</p> <p>Spread footing on soil – Extend borings the greater of 30 feet or two times the estimated footing width (2B) below proposed bottom of footing elevation. When rock is encountered, extend borings a minimum of ten feet into recoverable rock (greater than 80 percent recovery) or 15 feet of low recovery rock.</p> <p>Friction piles/piles bearing in soil – Extend borings 20 feet minimum below the estimated pile tip elevation, or a minimum of two times the maximum pile group dimension, whichever is greater, unless rock is encountered.</p>
Bridge (cont.)	<p>Drilled shafts bearing/socketed into bedrock – Extend borings a minimum of three shaft diameters below the estimated tip elevation, or two times the</p>

Area of Exploration	Boring Depth ^{1,2,3}
	<p>maximum shaft group dimension, whichever is greater, but not less than ten feet.</p> <p>Wingwalls – refer to Retaining Wall requirements.</p>
Culvert	<p>Box Culvert - Refer to Roadway Embankment requirements, but borings must extend a minimum of ten feet below the bottom of the culvert. If bedrock is encountered above, at or within five feet of the proposed bottom of the culvert, core a minimum of five feet of rock below the bottom of the culvert. In carbonate rock or mined areas, it may be necessary to core additional bedrock.</p> <p>Arch culvert or open bottom culvert (i.e., strip footings) – Refer to Bridge (spread footings on rock or soil) requirements.</p>
Retaining Wall	<p>Spread footing on soil – Extend borings a minimum of two times the height of the proposed wall, unless rock is encountered.</p> <p>All other retaining wall foundation types – Refer to appropriate Bridge foundation requirements.</p> <p>Linear walls (soldier beam and lagging, sheet pile, etc.) – Extend borings along face of wall a minimum of four pile diameters below the estimated shaft tip elevation, but not less than ten feet. For shafts embedded in bedrock drill ten feet minimum below tip in non-carbonate bedrock, and 20 feet minimum below tip in carbonate bedrock and deep mined areas. For sheet pile walls drill ten feet minimum below the estimated sheeting tip elevation.</p> <p>Soil and rock anchors – Extend borings a minimum of five feet below lowest elevation of anchor.</p>
Sound Barrier Wall	<p>Spread footing on soil – For isolated footings ($L < 2B$) extend borings a minimum of two times the footing width below the bottom of footing elevation. For continuous footings ($L > 5B$) extend borings a minimum of four times the footing width below the bottom of footing elevation. For footings with $5B > L > 2B$ use linear interpolation between depths of two and four times the footing width beneath the bottom of footing elevation.</p> <p>Spread footing on rock – See requirements for Bridge spread footings on rock, although 20 feet requirement for carbonate rock and mined areas may not be necessary if footings are lightly loaded.</p> <p>Drilled shafts in soil – Extend borings a minimum of four pile diameters, but not less than 10 feet, below estimated tip elevation.</p>
Sound Barrier Wall (cont.)	<p>Drilled shafts in bedrock - For non-carbonate rock extend borings ten feet below estimated tip elevation. For carbonate rock extend borings 20 feet below estimated tip elevation.</p>

Area of Exploration	Boring Depth ^{1,2,3}
Pavement Design	Extend borings minimum of three feet below the proposed subgrade elevation.
Dynamic Message Sign	<p>Spread footing on soil – Extend boring two times the footing width below the bottom of footing or 10 feet into competent soil (medium dense granular or hard cohesive), whichever is less.</p> <p>Spread footing on rock – See requirements for Bridge spread footings on rock, although 20 feet requirement for carbonate rock and mined areas may not be necessary if footings are lightly loaded.</p> <p>Drilled shaft – Extend boring a minimum of 20 feet below the top of drilled shaft into competent soil (medium dense granular or hard cohesive), or 10 feet into bedrock, whichever is less.</p>
Landslide	Extend borings beneath the failure plane, and ideally extend borings ten feet into bedrock. If bedrock is not encountered extend borings a minimum of 20 feet into competent soil (i.e., dense to very dense cohesionless soil and hard cohesive soil).

- Notes: 1. In general, borings should not be terminated in loose cohesionless soil, very soft or soft cohesive soil, fill, or organics. They should also not be terminated in voids, soil/clay seams, or coal seams in bedrock. Borings should be terminated in competent material that is believed to be stable from a settlement and stability perspective. Borings must also never be terminated at SPT refusal with an assumption that bedrock has been encountered. Unless the depth of the boring at SPT refusal satisfies information requirement for the various design elements the boring may serve, never assume that bedrock has been encountered. If it is important to know where top of competent bedrock is, continue the boring until the presence of bedrock can be verified. Where bedrock controls the boring depth criteria, a minimum bedrock core recovery of 80% should be obtained for the required depth.
2. In general, the boring depth criteria is applicable to all design phases since ideally borings performed during the Alternatives Analysis Phase and the Preliminary Design Phase will be used for final design. Consideration can be given to not advancing the borings to the depths indicated above for the Alternatives Analysis Phase only. Also, when considering the required depth of borings, be aware and recognize that any boring may serve the project design beyond the primary purpose of the boring (i.e., the specific design element or feature a boring is intended for obtaining relevant subsurface information). Borings may provide valuable information for more than one project design element, and have a cumulative impact of increasing both the overall accuracy of and confidence in the design model and the level of risk. In rare instances, the total depth of a boring may not be dictated by the needs of the specific design element that

dictates the location of the boring, but by a more global project need or requirement.

3. As mentioned in [Note 1](#), if soil borings are terminated before the minimum criteria is achieved because it is believed that bedrock was encountered, perform bedrock core drilling (five feet minimum) in an adequate number of borings to verify that bedrock, and not cobbles, boulders, or some other obstruction, was actually encountered.

Relative to overall design and construction costs, the subsurface exploration and associated laboratory testing is monetarily extremely small. Cost for this work is estimated in the range of two tenths of one percent (0.2%) of total combined design and construction cost. This is the cost of identifying and quantifying the most significant single unknown on any project involving material with the least quality control, the greatest variability, and that presents the highest risk for any design and construction venture. An adequately planned and executed subsurface exploration and laboratory testing plan provides a significant and cost effective approach to risk management for any highway design and construction project.

3.2.5 Contingency Test Borings

In situations where it is unclear as to the appropriate number, location, and depth of test borings, “contingency” borings should be included in the subsurface exploration program. These contingency borings should only be drilled if deemed necessary after drilling the primary test borings. For example, when drilling for a structure underlain by pinnacled bedrock (e.g., limestone) or extremely variably weathered bedrock (e.g., schist, gneiss, etc.), contingency test borings can be included on the Test Boring Plan between the primary borings, and shown on the Schedule of Proposed Borings. If the primary test borings indicate relatively uniform subsurface conditions, the contingency test borings may not have to be drilled. Conversely, if the primary test borings indicate a significant variation in the subsurface conditions at one or more of the proposed substructures, the contingency test boring(s) can be drilled. As another example, when drilling for a relatively small structure underlain by flat bedded stratigraphy, one of the two borings at each substructure could be labeled as contingency borings, and only drilled if the primary borings show that the subsurface conditions are not uniform across the site.

In many situations, it may be prudent to specify contingency borings as a matter of practice rather than only in areas of uncertainty as discussed above. Having contingency borings already built into an exploration program may save valuable design schedule and budget by not having to obtain a supplement. Alternatively, in situations where subsurface conditions are more uniform than anticipated, or in situations where uniform subsurface conditions are expected, the contingency borings can simply be deleted, or the footage applied to another location on the project where unanticipated needs develop.

It should be noted that test borings can be added during the subsurface exploration, such as, when unforeseen conditions are encountered by the scheduled borings; however, if uncertainty in the number and location of test borings exists during planning of the subsurface exploration program, it is preferred to include “contingency” test borings so that they can be

surveyed, discussed, coordinated, etc. in advance of actually performing the subsurface exploration program.

In any situation where contingency borings are included in a subsurface exploration program, it is imperative to have a mechanism established to assess when it is necessary to drill the contingency borings. This is simply a matter of good communication and planning. If contingency borings are included, timely assessment and decisions during the subsurface exploration program keeps the process efficient and coordinated, helping assure a better end product.

3.3 TEST PITS

Test pits are another option that should be considered for investigating subsurface conditions at a project site. Test pits should not be used in lieu of test borings, but instead should be used as a supplemental exploration method to test borings. Test pits are generally more appropriate to explore subsurface conditions for proposed roadways, but are sometimes useful for structure explorations. Test pits may be especially useful or beneficial for exploring shallow conditions or when it is suspected that an extensive zone of cobbles/boulders may be encountered. A test pit can easily and inexpensively clarify or verify subsurface conditions that may be important to design or construction. Publication 222, Section 211 provides guidelines and requirements for test pit operations.

Test pits are advantageous for numerous reasons, including:

1. Test pits expose a large area of the subsurface which often provides a better visual understanding of soil stratification compared to test borings
2. Test pits provide a good understanding of oversized material (e.g., coarse gravel, cobbles, boulders), which is difficult or cannot be sampled with test borings
3. Test pits are a good method for investigating uncontrolled fills, including construction debris
4. Test pits are relatively quick and inexpensive
5. Test pits provide an indication of ease of excavation
6. Test pits are a good method for investigating borrow sources and/or obtaining bulk samples for laboratory testing
7. Test pits can be used to determine depth to groundwater, including seasonal high groundwater level and infiltration rate, which are sometimes needed for stormwater design.

Test pits do have some shortcomings, including:

1. Test pits are limited to a relatively shallow depth, typically approximately 10 feet.
2. Test pits generally cannot be advanced into bedrock.
3. Test pits may not be able to be advanced below the groundwater level, particularly in cohesionless soils.
4. Test pits create a relatively large area of disturbance, and require additional area to temporarily stockpile excavated material.

5. Terrain must be relatively flat for test pit equipment to access.
6. If the exploration requires an individual to enter a test pit they are subject to Occupational Health and Safety Administration (OSHA) regulations. Depending on soil conditions, depth of test pit, groundwater conditions, and surcharge loads, an excavation support protection system may be required if excavation slopes cannot be laid back in accordance with OSHA regulations.

3.4 EXAMPLE PROJECTS

As previously indicated, exact boring requirements (i.e., locations and numbers) cannot and should not be specified in this publication due to the multiple unknowns and variables involved with any given project, and the variability of geologic conditions. Keeping this in mind, below are examples of typical subsurface exploration programs for Department roadway and bridge construction projects. These typical scopes must be considered within the context of project specific needs, complexity, risks, and geologic conditions, in determining the subsurface exploration requirements for a specific project.

1. Final Design Phase for a box culvert replacement with no roadway widening or vertical profile change: box culvert is 75 feet long, new box is approximately same size as existing, wing walls are 15 feet long and new pavement will match existing.
 - Since the culvert is less than 100 feet long, and wingwalls are less than 25 feet long, a minimum of two test borings are required. A boring must be drilled at the inlet and outlet of the culvert.
 - Consider a contingency test boring located between the two primary test borings if there is indication that the subsurface conditions may not be relatively uniform beneath the proposed culvert. Drill the contingency test boring after the two primary test borings are drilled and only if needed to better define the subsurface conditions.
 - Drill test borings to depth of two times the embankment height or deeper until dense/hard material of sufficient thickness is encountered.
 - Include at least one pavement core to verify the existing pavement section.
2. Final Design Phase for a two-span bridge replacement with minor approach embankment widening: the bridge is 120 feet long; spans a roadway (pier in the median), approximately 35 feet wide, wing walls are 20 feet long, existing foundations are unknown, 50 feet of minor approach embankment widening on each side of the bridge, 15-foot high approach embankments, five feet (max.) widening on each side of embankment, and new pavement to match existing pavement section.
 - Drill two borings at each substructure unit. These should extend ten feet into bedrock. If bedrock is deep, say more than approximately 50 feet below original ground surface (i.e., not roadway embankment surface) and overburden soil is granular and dense to very dense, consideration can be given to not advancing borings into bedrock. This assumes friction piles and/or spread footings on soil will be used. However, it is always good practice to advance borings into rock to

have the needed information to be able to consider deep foundations supported on bedrock. In a situation where rock is very deep, and near-surface soils are inadequate for a spread footing, it is important that borings be conducted to a sufficient depth to assure that adequate resistance can be mobilized for friction piles, or to a depth necessary to adequately assess alternate ground modification or improvement methods.

- Drill one boring on each side of the bridge (two borings total) near the toe of the existing embankment in the widening areas. Extend these borings approximately ten feet below ground surface to determine if undercutting is needed prior to widening embankment.
- Obtain one pavement core on each side of the bridge.

3.5 CONSIDERATIONS WHEN PLANNING SUBSURFACE EXPLORATION PROGRAM

3.5.1 Mandatory Pre-bid Meeting

During preparation of the SBSTC, the PGM and DGE must decide if a Mandatory Pre-Bid Meeting will be required. If it is determined that a Mandatory Pre-Bid Meeting is necessary or prudent, this requirement, along with specifics regarding date, time, and location of the meeting, must be included in the SBSTC bid documents. The Mandatory Pre-Bid Meeting is addressed in Article I of the Instruction to Bidders in the SBSTC located in Publication 222. When a Mandatory Pre-Bid Meeting is required, drilling contractors must attend in order to be eligible to submit a bid to perform the subsurface exploration program.

The need for a Mandatory Pre-Bid Meeting must be determined on a project by project basis. The main purpose of a Mandatory Pre-Bid Meeting is to ensure that the drilling contractors understand the subsurface exploration requirements of the project so they can bid the work accordingly. The boring locations must be staked/marked before the meeting is conducted. In situations where it is absolutely not practical or feasible to stake boring locations prior to the meeting, the location of each boring must be clearly identified in the field during the meeting. Minutes of the meeting must be prepared by the PGM and distributed to all attendees. The meeting minutes become part of the SBSTC.

A Mandatory Pre-Bid Meeting should be considered for projects that include:

- A large quantity of work, multiple drill rigs or an aggressive schedule
- Non-standard work items (e.g., instrumentation, in-situ testing other than SPT, borehole camera work, etc.)
- Boring locations that are difficult to access, require considerable private property coordination, are located with railroad right-of-way (ROW), or require significant maintenance and protection of traffic
- Sensitive environmental considerations (e.g., potential subsurface contamination, wetlands, high quality waterways, etc.)
- Considerable maintenance and protection of traffic (MPT) requirements or MPT requirements for difficult conditions or high volume roadways

- Performing work under a Health and Safety Plan (HASP)
- Potentially unsafe drilling conditions.

Conversely, a Mandatory Pre-Bid Meeting is typically not necessary for projects that include:

- “Standard” work items (e.g., split-barrel sampling, SPT, rock core drilling, pavement cores, auger borings, etc.)
- Readily accessible test boring locations, no or limited private property owner coordination, or no work on railroad ROW.
- No or limited maintenance and/or protection of traffic.

3.5.2 Subsurface Exploration Capabilities of the Department

The Geotechnical Section of the Bureau of Project Delivery, Construction Materials Division (i.e., Central Office Geotechnical Section), has capabilities and equipment that can be used to augment traditional subsurface exploration techniques. Equipment includes:

- Piezocone penetrometer testing (CPT) equipment
- Down hole camera for viewing voids in bedrock and other subsurface features
- Laser cavity surveying system for “large” void mapping
- Vertical and horizontal inclinometers
- Vibrating wire piezometers (standard and drive point)
- Borehole vane shear test (VST) equipment

The PGM and DGE should consider the usefulness of this equipment when planning the subsurface exploration program. If deemed useful, contact the Central Office Geotechnical Section staff to discuss further and make arrangements for using the equipment.

3.5.3 Utilities

Utilities are a major concern when performing excavations, which includes test borings and test pits. The PGM, Department, and drilling contractor have responsibilities regarding locating above and below ground utilities prior to performing a subsurface exploration program. A design phase PA One Call must be made by the designer (i.e., Department for in-house projects, otherwise consultant) early in the project. This task is usually performed by the roadway or bridge designer. Utility information obtained must be presented on the Test Boring Location Plan, and any utility information observed during site visits should be added if not already shown. Prior to any drilling activities, including test borings and test pits, the drilling contractor is also required to perform a PA One Call. Currently, PA One Call requires a minimum of three days advance notice prior to the start of drilling, and the borings/excavations must be clearly marked in the field with white paint. A stake with white flagging attached or painted white is acceptable. Note that PA One Call does not locate Department owned utilities as the Department does not subscribe to PA One Call. Department owned utilities must be cleared by the Department. The DGE can provide guidance as to how this is accomplished. Similarly, other privately owned utilities may not be located by PA One Call and must be located by the owner of the utility or by a utility locating contractor.

Consideration must be given to the location of utilities when preparing the subsurface exploration program. When preparing the Test Boring Location Plan, locate borings as needed to gather the necessary subsurface information but as far away from utilities as possible. Borings must be located a minimum of 10 feet away from overhead utilities, regardless of whether the overhead utility contains power. In accordance with Publication 222, Section 103.10, the PGM (or DGE for in-house projects) must contact the utility company if overhead power lines are in the vicinity of the proposed borings. For voltage to ground under 50kV, the minimum clearance is ten feet. For voltage to ground over 50kV, the minimum clearance is ten feet plus four inches for every 10kV over 50kV. If these minimum clearances cannot be met, the drilling contractor must make special arrangements with the utility company to permit the work to be completed (e.g., de-energize line, shield line, etc.). Note that in most cases 20 feet of clearance between test boring equipment/tools and an overhead utility is sufficient per OSHA regulations.

3.5.4 Health and Safety Plan (HASP)

Publication 222, Section 103.09(d), discusses requirements for performing the subsurface exploration under the direction of a Health and Safety Plan (HASP). A HASP must be prepared for subsurface exploration programs where known or suspected site hazards exist. The subsurface exploration work must be performed in accordance with the HASP.

The drilling contractor is responsible for the health and safety of its employees and any subcontractor's employees; therefore, the contractor must develop a site-specific HASP for the project. As indicated in Publication 222, Section 103.09(d), the site-specific HASP must be submitted to the Department's Environmental Manager for review and acceptance. The consultant/Department must provide the contractor with information regarding the known or suspected site hazards; however, it must be made clear that other hazards may exist. This information can be provided as a narrative in Attachment II of the SBSTC.

The consultant and/or the Department must prepare separate HASP's for their employees to follow while on site performing drilling inspection or other tasks. A generic HASP is provided in the appendix of Publication 222 that indicates the general framework of a HASP. This is generic and must be modified/amended as necessary to address site specific conditions and concerns. As per Publication 222, Section 103.09(e), if potentially contaminated material is encountered during the subsurface exploration that was not previously suspected, the exploration operation must be halted in a safe and controlled manner. The PGM must immediately notify the DGE, and the DGE must contact the District Environmental Manager.

3.5.5 Maintenance and Protection of Traffic (MPT) and Time Restrictions

Where possible, test borings should be located off the traveled roadway and shoulders in order to avoid the need for maintenance and protection of traffic. Test borings located on the roadway or shoulder pose a risk to motorist and worker safety, and MPT can be a costly item for the drilling contractor to provide. However, in many situations test borings located on the roadway or shoulder are needed in order to obtain the necessary subsurface information for a project.

Publication 222, Section 216 provides general requirements for MPT. Site specific MPT requirements must be added to the SBSTC to indicate the specific boring(s) where MPT is required/anticipated, and to indicate which Pennsylvania Typical Application (PATA) drawing from Publication 213 applies to each boring. Attachment 1, Schedule of Proposed Borings must indicate which borings require MPT by including the designation “MPT” in the column titled “OTHER”. Attachment II can be utilized to list which traffic control scenario/figure from Publication 213 applies to each boring. The Department traffic/highway engineer must review and approve the proposed MPT requirements for the test boring plan and any subsequent modifications to the plan.

In addition to MPT requirements, time restrictions sometimes apply to test borings located on the roadway or shoulder. Time restrictions typically apply to interstates or heavily traveled roadways. Drilling is sometimes prohibited during peak travel hours, on holidays, and days surrounding the holidays when travel volumes are typically high. There may be situations when drilling is only permitted at night and early in the morning. The Department will determine if time restrictions apply to any of the test borings, and the consultant/Department must include these restrictions in the SBSTC. Attachment II can be used to present these requirements. Additional restrictions may be necessary to prohibit boring operations during inclement weather such as fog, heavy rain, snowstorms, and freezing rain, or periods immediately after snow and/or freezing rain events when emergency vehicles and snow/ice removal operations are active and equipment requires clear unobstructed access to the roadway.

3.5.6 Minimum Number of Required Drill Rigs and Allotted Time to Complete Work

The minimum number of drill rigs required for a project must be indicated in the SBSTC, Subchapter 5A “Instructions to Bidders”, Article C. Either the specific number or a range in number of drill rigs required must be stated in Article C of the contract. Additionally, the number of calendar days allotted to complete the work must be provided in the Contract Agreement (i.e., Publication 222, Subchapter 5D – Contract Agreements, Item 3). The required number of drill rigs will be a function of the project schedule, the estimated drilling footage, the estimated production rate (footage drilled per shift) and other controlling factors.

- The overall project schedule is the first consideration when estimating the number of required drill rigs particularly when the subsurface exploration information is needed to complete the geotechnical tasks. In many cases, the geotechnical work must be completed fairly early on in a project since other disciplines rely on the geotechnical recommendations/considerations.
- For smaller projects, such as single to several span structures and/or minor roadway projects, one drill rig will be adequate in most cases. For larger projects, like numerous span structures and/or major roadway projects, multiple drill rigs will typically be needed.
- Projects where the estimated drilling footage is in the range of 500 feet or less can generally be conducted with one drill rig. Projects with more than approximately 500 feet of estimated drilling should consider the use of more than one drill rig.

- Numerous considerations must be made when estimating the time it will take to complete the subsurface exploration. Typically, it is reasonable to estimate that each operating drill rig will complete 30 to 40 linear feet of drilling per eight-hour work shift. This may be higher in weaker soils or rock, or lower in denser or more granular soils or harder rock.
- Situations that reduce the per shift production rate of a drill rig include: limited working hours due to traffic or other restrictions, deep soil sampling and SPT, obstructions (e.g., cobbles, boulders, demolition fill, etc.), denser soils, pinnacled bedrock, voids in bedrock, thick zones of highly weathered rock, very hard rock formations, difficult access to/between borings, borings drilled in/on water, borings requiring railroad flagging/coordination, inclement weather, and more. “Deep” wireline rock core drilling, say 30 feet or more, in individual borings can help increase the per shift production rate of a drill rig, along with shallow overburden and truck accessible test borings.
- In addition to the production rate of the drill rig, time estimates for completing subsurface explorations must account for mobilization (and demobilization), grouting of borings, instrumentation installation, equipment repairs, and more.

3.5.7 Temporary Potable Water Supply

Potable water sources including wells, springs, or other ground water supplies used for human consumption can be impacted by drilling. Consideration must be given to the location and type (i.e., shallow uncased, deep cased, etc.) of these water sources when preparing the subsurface exploration program. When preparing the Test Boring Location Plan, locate borings as needed to gather the necessary subsurface information, but as far away as possible from potable water sources. If the PGM/DGE determine there is a possibility that potential impact by drilling could occur, the Boring Contract should be amended to alert the contractor to be prepared to provide a temporary potable water supply. Publication 222, Section 219 provides general guidelines and requirements for hooking up, maintaining, and disconnecting a temporary potable water supply for impacted property owners.

3.6 TEST BORING EQUIPMENT, TOOLS, AND PROCEDURES

3.6.1 Drill Rigs/Machinery

Drill rigs/machinery used to perform test borings are mounted on a variety of equipment to allow access to most locations. Truck mounted drill rigs are the most efficient when borings are located on relatively flat ground and are easy to access. Drill rigs mounted on tracked and all-terrain vehicles/machinery are necessary when borings are located on uneven terrain, in shallow (less than a few feet) water, and/or when access to boring locations is difficult. Drill rigs mounted on skids are useful on steep terrain where winching may be necessary to reach the boring location, or where the drill equipment must be supported on cribbing or a benched slope. Skid mounted drill rigs are typically smaller than other types of drill rigs so they are also useful in tight spaces, and they can be set in place by a crane. A drill rig placed on a barge is necessary for drilling in more than a few feet of water, and depending upon the size of the barge any size/type drill rig can be used.

It is the contractor's responsibility to select the type of drill rig used to perform the work. Selection of equipment will depend not only upon the location of the borings, but also upon the equipment owned/available to the contractor, depth of proposed borings, auger/casing requirements, in-situ testing requirements, instrumentation installation, etc. During field reconnaissance, potential boring access and equipment needs should be considered and discussed in the Subsurface Exploration and Planning Submission. Generally speaking, borings accessible with a truck mounted drill rig will be less expensive than borings that require an off-road type drill rig. Therefore, when possible and as long as the required/necessary subsurface information can be obtained, borings should be located so that they are accessible with a truck mounted drill rig.

Publication 222, Subchapter 5E "Standard Specifications for SBST Contracts", Section 201 addresses mobilization of drilling equipment. Mobilization (and demobilization) includes all work necessary to move to and from the site. Mobilization (and demobilization) includes movement of drill rig(s), tools, personnel and materials to and from the project site required to complete the work. Mobilization also includes costs associated with obtaining permits and insurance, site restoration and clean up, and any other items required to complete the work. Mobilization for drilling equipment is a lump sum item, is independent of the number of drill rigs required by the contract, and is independent of whether the borings are located on land or in water. Mobilization does not include movement between borings, or work associated with accessing borings (e.g., vegetation/tree clearing, site leveling, guide rail removal, etc.). The cost of these must be included in the per foot price for advancing the borings.

3.6.2 Test Boring Advancement and Support in Soil

There are several methods to advance test borings and support boreholes in soil to permit the desired sampling and testing. These include hollow-stem or solid flight augers, flush thread steel casing, and mud rotary. Borehole advancement and support must be done in accordance with the requirements of Publication 222 and ASTM standards, where required by Publication 222.

The method of borehole advancement must be indicated in the SBSTC in Attachment 1, Schedule of Proposed Borings. The advancement method is typically selected by the drilling contractor (i.e., indicated as "NS") unless a specific method is desired by the Engineer and within the requirements of Publication 222. Caution should be used when requiring a specific method of advancement and support, because if this method is unsuccessful, it may result in additional costs for the contractor to supply alternate equipment to complete the work. Payment for borehole advancement and support in the SBSTC is incidental to the cost of soil sampling.

The soil boring diameter must also be specified in Attachment 1 of the SBSTC. Typically, the minimum required inside diameter of the augers or casing, or hole diameter if using drilling mud, is approximately 3.5 inches. This diameter will permit standard split-barrel sampling, NX or NQ rock core drilling, and undisturbed sampling with 3-inch diameter thin walled samplers/tubes. If a larger hole diameter is needed due to sampling, instrumentation, etc. requirements, it must be specified in Attachment 1.

3.6.2.1 Hollow-Stem Augers

In Pennsylvania, test borings are typically performed using continuous flight hollow-stem augers. Hollow-stem augers can be advanced through most soils encountered in Pennsylvania; however, advancement with hollow stem augers is often difficult in soils containing obstructions, such as cobbles, boulders, and demolition debris. Hollow-stem augers cannot be advanced through bedrock unless it is very soft and/or highly weathered, and even in these rocks it can be difficult and slow. Hollow-stem augers provide support of the borehole through soil and permit sampling and testing to be performed at the desired depths. Augers typically come in 5-foot-long sections, have a cutter head on the lead/bottom auger, and are joined by mechanical (typically bolts) connections.

Advancing borings with augers should be done without introducing water into the augers. Groundwater, including perched areas, can be more accurately detected when water is not used to advance the boring. Additionally, the natural moisture content of the soil, particularly coarse grained soil, is not altered by the introduction of drill water. Lastly, augering without water is typically the quickest and least costly method of advancing borings.

A mechanical center/pilot bit assembly is used inside of the augers during advancement to prevent soil from entering the augers. In some soils, a “natural plug” may form at the bottom of the auger during advancement that may prevent disturbed soil from entering the augers. However, as indicated in Publication 222, it is not acceptable to rely on a natural plug because its formation is unpredictable and sometimes difficult to measure/detect. Consequently, a center bit assembly **MUST BE USED** when advancing the hole with hollow-stem augers.

If disturbed material enters the augers after removal of the center bit assembly, a roller bit or other cutting tool along with water, if approved by the PGM, can be used to flush disturbed material from the inside of augers to permit sampling at the desired depth. Extreme caution and care must be used when cleaning the augers with water so as not to disturb the soil below, and this practice should only be used when absolutely necessary. Once the augers reach the top of rock, rock core drilling can be performed through the augers.

Although hollow-stem augers are versatile, they do have limitations. They can advance through most soils, but they may obtain refusal (i.e., stop advancing) in:

- Very dense soil, like some glacial tills
- Cobbles/boulders within the overburden or in embankment fill
- Fill materials with obstructions such as concrete or other construction debris.

Additionally, where rock must be penetrated to sample softer (soil) material below, like in carbonate geology or deep mined areas, hollow stem augers typically cannot be used. Additionally, when hollow-stem augers encounter pinnacled bedrock, they often “wander” or “walk”, preventing soil sampling and rock core drilling tools from being inserted through the augers. Consequently, the hole may have to be abandoned and redrilled. In these situations, rotary drilling with casing or mud will most likely be required.

3.6.2.2 Drilled and Driven Steel Casing

Steel casing is another method used to advance test borings and support boreholes. Similar to hollow-stem augers, sampling and testing can be performed through the casing at desired depths. Casing typically comes in 5-foot-long sections, is joined by threaded ends, and is equipped with either a drive or cutting shoe on the lead/bottom piece of casing. Casing is typically advanced by the rotary method, but it can also be advanced by driving with a drop hammer. A casing advancer, roller bit, or other cutting tool along with water is used to flush disturbed material from the inside of the casing to permit sampling at the desired depth. Bits which discharge water from the bottom or otherwise disturb the soil at the top of the sampling interval are not permitted. Casing advanced by the rotary method and equipped with an appropriate cutting bit can be used to advance through cobbles, boulders, obstructions, and bedrock. The rotary method of advancement is much more efficient/quicker compared to advancement with a drop hammer. Additionally, casing is often more effective than hollow stem augers when advancing borings in pinnacled bedrock. Once casing refusal is met at the top of bedrock, rock core drilling can proceed through the casing.

3.6.2.3 Mud Rotary

Mud rotary drilling is also an option to advance test borings and support boreholes, although a much less common method in Pennsylvania compared to hollow-stem augers and casing. This method relies on drilling fluid (i.e., mud), which is heavier than water, to support the uncased borehole. Sometimes casing or augers, in addition to mud, are used in the upper part of the boring above the groundwater to help maintain borehole stability. Drilling mud typically consists of water mixed with powdered bentonite (clay). Other additives that are not toxic or potentially harmful, including polymers, can be used to create or enhance the drilling mud. The borehole is advanced by using a cutting bit, often a roller bit, attached to drill rods. Mud is circulated through the drill rods during advancement to keep the borehole filled/supported and to carry the cuttings to the surface. Once the desired depth is reached, the drill rods are removed from the borehole and the sampling and testing equipment is inserted through the mud. Once the top of rock is encountered, rock core drilling can be performed through the drilling mud.

The mud rotary method is often used when drilling in deep, coastal plain deposits below groundwater. The use of mud eliminates the need to advance and retrieve long strings of augers/casing, which can be very time consuming and labor intensive. Preparation/start-up for mud rotary drilling is typically more time consuming compared to augers and casing, and, therefore, generally not advantageous for relatively shallow overburden conditions that are often encountered in Pennsylvania. The use of drilling mud can cause environmental concerns including disposal of the mud and/or sediment contamination of waters, but these concerns can typically be addressed with on-site containment and treatment. The use of drilling mud can also be used in conjunction with augers or casing to help control artesian conditions or running sands/blow-ins.

3.6.2.4 Solid Stem Augers

Solid stem augers have limited use for subsurface explorations. The main use for solid stem augers is to collect bulk, disturbed soil samples from relatively shallow depths (i.e., ten feet or less). At greater depths, it is difficult to determine the actual depth from which the soil cuttings originated, and if groundwater is encountered, the cuttings are often not representative of the in-situ conditions. Refer to Publication 222, Section 217 for requirements for bulk sampling with solid flight augers. Since the augers are solid, soil sampling and testing tools cannot be inserted through the augers like they can with hollow-stem augers. If SPT's or other sampling methods are required, the solid stem augers must be removed from the hole, leaving it unsupported. Unsupported boreholes will typically collapse at relatively shallow depths, particularly if groundwater is encountered, and they are generally prohibited. Therefore, solid stem augers are generally not permitted to be used to advance a boring in soil when sampling, testing, rock core drilling, instrumentation installation, or any other procedure requiring an open hole is needed. Another use for solid stem augers is to probe for the top of bedrock when coring of the bedrock is not required, however, caution must be used when determining top of rock with this approach, as the "apparent" top of rock could be a cobble, boulder, or other obstruction.

3.6.3 Test Boring Soil Sampling

As previously indicated, one of the main goals of the subsurface exploration is to obtain samples of the various soil strata underlying the site for visual classification and laboratory testing. Soil samples obtained from boreholes are considered either disturbed or undisturbed. Disturbed soil samples are collected in a way that alters the in-situ state of the soil, whereas undisturbed samples are collected to maintain as much as possible the in-situ state of the soil. Methods used to obtain disturbed soil samples include split-barrel sampling and bulk sampling with augers, and a method used to obtain undisturbed soil samples is the thin-walled tube (Shelby tube) sampler.

3.6.3.1 Split-Barrel Sampling and Standard Penetration Testing (SPT)

The most common method in Pennsylvania of obtaining disturbed soil samples is split-barrel sampling, sometimes referred to as split-spoon sampling. Split-barrel sampling, in conjunction with Standard Penetration Testing (SPT), is an ideal method to: visually classify soil, estimate soil density/consistency and moisture content, and collect disturbed samples for laboratory testing. In addition to these, this method is commonly used because the equipment is readily available and rugged, the procedure is relatively easy, and nearly all soils and some very weak rocks can be sampled/tested. Split-barrel sampling and SPT must be performed in accordance with Publication 222 and ASTM D 1586 (where Publication 222 allows) for Department projects.

It should be noted that extreme care must be taken when performing SPT in order to achieve the best quality and most representative data possible. The specifications for performing the testing must be carefully and closely followed because numerous items affect the results. These items include: preparation of the borehole; condition of the sampler, hammer and drill rods; type of hammer system used, number of rope wraps around the cathead, and distance the hammer is dropped; and proper measurement of 6-inch increments and recording of hammer blows. It is also important that the boring log clearly describes the hammer system used to

perform the standard penetration testing in order to correct the recorded N-values for hammer efficiency in cases where N-values are used to select soil parameters.

The split-barrel sampler consists of a hollow steel barrel. For Department projects, the required barrel length is 24 to 30 inches, and the inside and outside diameters of the sampler are 1.375 and 2.00 inches, respectively. The barrel is split in half lengthwise, and the ends are threaded. An open/hollow steel shoe is threaded onto the bottom of the tube, and the top threads of the barrel are used to attach the sampler to the drill rods. A schematic of the sampler is shown in ASTM D 1586.

Split-barrel sampling and SPT are performed by advancing the test boring to the desired depth by using one of the methods (i.e., hollow-stem auger, casing, or mud) discussed [above](#). Note that the first sample and SPT typically begin at ground surface. Once the desired depth is achieved and all drill cuttings are removed from the boring, the sampler is attached to steel drill rods and lowered to the bottom of the boring in a controlled manner (e.g., not dropped, free-fall, etc.). The sampler is advanced 1.5 feet into the undisturbed ground starting at the bottom of the borehole with a 140-pound hammer dropped from a height of 30 inches. The number of hammer blows to advance the sampler each of the three, six-inch increments (i.e., 1.5 feet total) constitutes the Standard Penetration Test (SPT), and these numbers are recorded on the boring log. The sampler is then removed from the borehole, and the split-barrel sampler is opened to remove the soil sample for visual classification and storage.

The recorded “N” value, which is the number of hammer blows to advance the sampler per foot, is the sum of the last two six-inch intervals for the three sample increments. The blows required to advance the sampler the first six inches are considered “seating blows” and are not included in the determination of “N” value. The correction of “N” values for hammer efficiency is discussed in Publication 222, Section 3.6.2. If a situation occurs when it becomes necessary to acquire a two-foot sample, the recorded “N” value is the sum of the middle two six-inch intervals for the four sample increments.

Split-barrel sampling and SPT can be performed continuously or at intervals. Continuous sampling/SPT should typically be used on Department projects. Continuous sampling/SPT means sampling and testing are performed every 1.5 feet without any gap/space between the bottom and top of consecutive samples. Interval sampling means there is a gap/space between the bottom and top of consecutive samples. A more detailed discussion on this is provided later in this chapter. Additionally, specific guidance regarding SPT refusal criteria and when rock core drilling should be performed are provided in Publication 222, Section 202.

Steel or plastic traps can be inserted between the shoe and barrel of the sampler if sample recovery is poor. Traps are typically advantageous for very loose to loose cohesionless soils but can be used anytime sample recovery is poor. These traps should only be used when needed, and when used it must be noted on the boring log because traps can influence (i.e., increase) SPT blow counts.

3.6.3.2 Bulk Soil Sampling with Augers

Another method for obtaining disturbed soil samples is bulk soil sampling with augers. This is a relatively inexpensive method to obtain disturbed samples of soil for visual or lab classification at relatively shallow depths. Additionally, sampling with augers provides a larger (bulk) sample which is needed for certain laboratory tests, including moisture density relation (compaction) and California Bearing Ratio (CBR). Bulk sampling with augers can be performed using either solid stem augers or hollow stem augers equipped with a center/pilot bit assembly. Bulk sampling must be performed in accordance with Publication 222, Section 217 and applicable ASTM standards.

There are several limitations/disadvantages to bulk sampling with augers. Soil sampling with augers can only be done to a limited depth, typically in the range of 10 feet. Past that depth, it becomes difficult to determine the specific depth at which samples were actually obtained. Another limitation of bulk sampling with augers is, generally, it is not successful below ground water, particularly in cohesionless soil. Once saturated conditions are encountered with augers, the resulting soil samples are generally too wet for visual classification or sample collection. Sampling with augers also does not provide a quantitative measure of the in-situ density/consistency of the soil like SPT provides. Lastly, the friction generated from advancing the augers, particularly in dense/hard soil can generate significant heat which can alter the moisture content of samples compared to the in-situ state.

3.6.3.3 Thin-Walled (Shelby) Tube Soil Sampling

Unlike the disturbed sampling methods previously discussed, undisturbed sampling methods are intended to preserve the in-situ nature of the soil to the best extent possible. Undisturbed soil samples are necessary when laboratory testing is needed to estimate in-situ soil properties. These in-situ properties most commonly include shear strength and consolidation but also could include unit weight and permeability. Undisturbed soil sampling is typically performed in fine grained, cohesive soils that are very soft to stiff (i.e., SPT N-values less than 15 blows per foot). Consolidation and shear strength of these soils often have a significant impact on projects, and estimating these soil properties is most commonly done by laboratory testing since they cannot be estimated with SPT. Other in-situ testing methods including vane shear and cone penetrometer, can be used to estimate shear strength and consolidation properties, but these methods are not used as frequently as laboratory testing.

Cohesionless soil samples cannot be obtained using undisturbed sampling methods because they lack the cohesion necessary for the soil to remain in the sampler during extraction from the borehole. Undisturbed samples of cohesionless soils generally are not needed because consolidation settlement is not a concern, and elastic settlement and shear strength properties can be reasonably estimated from SPT. Shear strength properties can be estimated with SPT in combination with lab classification testing for preliminary design and non-critical applications. Disturbed samples can usually be used (i.e., remolded) to perform laboratory direct shear testing to obtain representative estimates of shear strength of cohesionless soils for more complex conditions or higher risk applications or situations.

The most common method in Pennsylvania to obtain undisturbed soil samples is using a thin-walled sampler, often referred to as a Shelby tube. Shelby tube samples must be obtained in

accordance with Publication 222, Section 203 and applicable ASTM standards. Shelby tubes made of 16-gauge brass, hard aluminum, or 16 or 18-gauge seamless steel, with a minimum length of 30 inches are required for Department projects. Sampler outside diameters ranging from two to five inches are available, but the 3-inch diameter is the most common and must be used for Department projects unless otherwise required and approved by the DGE. Three-inch diameter samplers are preferred because they are readily available, fit inside standard augers/casing that are typically used to advance borings, and are ideal for laboratory testing, including consolidation, direct shear and triaxial shear.

Preferably, undisturbed samples should be obtained in a boring offset three to five feet from a previously drilled SPT boring. By doing an SPT boring first, the depth of the material to be sampled with the Shelby tube will be well defined, which should improve the likelihood of obtaining a quality undisturbed sample. Consideration should be given to performing SPTs in the offset boring above and below the undisturbed sample to ensure the material type and consistency is the same as encountered in the adjacent SPT boring. Obtaining an undisturbed sample in the SPT boring (i.e., not an offset boring) can be considered when the material to be sampled is at considerable depth and SPT testing has been performed to the extent that the material thickness/depth is well defined.

When undisturbed samples are required for a project, it is recommended that at least two be obtained for each soil stratum being sampled, and possibly more depending upon the number and/or type of tests proposed or anticipated. Consolidation and direct shear testing require fairly small samples, so one Shelby tube sample with good recovery and of good quality will be adequate to perform one set of these tests. However, triaxial shear and unconfined compression tests require considerably more sample. For example, a three-point triaxial shear test will require a good quality Shelby tube sample with nearly full recovery (e.g., 24 inches). Therefore, when preparing the subsurface exploration plan, consideration must be given to the number and type of laboratory tests that may be performed to estimate the number of Shelby tube samples that should be obtained.

A minimum of two Shelby tubes is recommended regardless of the number and/or type of tests proposed because the quality of the sample recovered is not known until it is extruded in the laboratory. Sometimes a sample recovered in a Shelby tube is believed to be of good quality based upon field observations, but upon extruding in the lab, the sample is found to be of poor quality or is not the material type that was expected. Therefore, it is always prudent to obtain more tubes than what is anticipated to be needed for laboratory testing.

A Shelby tube sample is obtained by advancing the boring with hollow-stem augers, casing or mud, and removing disturbed soil to the desired sampling depth. The sampler is attached to the drill rods and lowered to the bottom of the boring. The Shelby tube is pushed into the undisturbed soil using the drill rig hydraulics. The sampler cannot be rotated or driven because this can cause significant disturbance to the soil being sampled and result in erroneous laboratory test results. Once the sampler has been pushed to the desired depth it must remain in place for a minimum of 15 minutes. Prior to removing the sampler from the borehole, the drill rods should be manually rotated two full revolutions to shear the soil at the bottom of the sampler.

Preserve and label the Shelby tube in accordance with the technical provisions of Publication 222, Section 203, which includes sealing both ends of the tube with wax to maintain the natural moisture content of the sample. Proper preservation and transportation of Shelby tubes are absolutely critical to obtaining good quality and reliable laboratory test results. Once preserved, the Shelby tube should be stored in an upright position with the top of the sample up (i.e., the same orientation as in-situ condition). Tubes must be kept in a location free of disturbance/vibration and must be protected from freezing or extreme heat. Do not store tubes in a vehicle. Deliver Shelby tubes to the laboratory as soon as possible. In order to obtain laboratory test results that are reliable and most representative of in-situ conditions, Shelby tube samples should be extruded and tested as soon as possible.

3.6.3.4 Piston Tube Sampler

A variation of the Shelby tube sampler is the piston tube sampler. The piston tube sampler is typically used in very soft to soft, wet, cohesive soils that cannot be retained in a standard Shelby tube sampler. Literature also indicates that this sampler can be successful at retrieving samples of very loose to loose, cohesionless soils, like non-plastic silts and sands.

In general, the piston tube sampler consists of a Shelby tube equipped with a piston and small diameter threaded piston rods that extend through the drill rods and connect the piston to the drill rig. The piston fits snugly in the Shelby tube, which creates a vacuum as the Shelby tube is pushed into the in-situ soil and as the sampler is removed from the borehole. This vacuum improves the likelihood of recovering an undisturbed sample compared to the standard Shelby tube. Additionally, the piston keeps drilling fluid and soil cuttings out of the Shelby tube while the sampler is lowered to the bottom of the borehole. The same sample preservation, storage, handling and transportation procedures used for the standard Shelby tube must also be used for the piston tube samples.

There are two basic types of piston tube samplers: the fixed piston sampler, and the free or semi-fixed piston sampler. Both types use the same equipment described above, and both types use a piston that is fixed in place at the bottom of the Shelby tube while the Shelby tube is lowered to the bottom of the hole. As previously mentioned, this fixed piston at the bottom of the Shelby tube prevents drilling fluid and soil cutting from entering the Shelby tube as it is lowered into the borehole. The difference between the two samplers is while the Shelby tube sampler is advanced into the in-situ soil at the bottom of the borehole, the piston is held fixed/stationary (i.e., does not move with the Shelby tube sampler) with the piston rods when the fixed piston sampler system is used, whereas the piston is free to travel/ride on top of the sample when the free/semi-fixed piston sampler system is used. If the free/semi-fixed piston sampler is used, the soil to be sampled must be of sufficient strength/consistency to push the piston upward as the Shelby tube sampler is advanced. In most cases, if a standard Shelby tube sample cannot be obtained because the soil is too soft, the fixed piston sampler is most likely the best alternate method to try to retain a sample.

The piston tube sampler is not commonly used for Department projects, and neither Publication 222 nor ASTM has a specification for sampling with a piston tube sampler. The U.S.

Army Corps of Engineers (USCOE) EM-1110-1-1804 does provide guidance for using this type of sampler. If a piston tube sampler is specified for use on a Department subsurface investigation project, a modification to the standard SBSTC specifications must be made to provide the contractor directions/requirements for performing this work.

3.6.3.5 Denison and Pitcher Tube Samplers

If relatively undisturbed samples of hard cohesive soils, soft rock, heavily cemented soils, and stratum containing gravel are needed, which cannot typically be sampled with a Shelby tube, a Denison or Pitcher tube sampler should be considered. Samples obtained from these types of samplers should be considered partially disturbed and not undisturbed, although these samples are often used for laboratory tests that are typically performed on undisturbed soil samples. The same sample preservation, storage, handling, and transportation procedures used for the standard Shelby tube must also be used for the Denison and Pitcher tube samples.

The Denison sampler is a double-tube core-barrel type sampler, although an additional inner liner (i.e., triple tube) can be used to aid sample handling. Denison samplers can generally either obtain four or six-inch diameter samples that are two feet long. The sampler consists of an outer barrel equipped with carbide cutting teeth, and a smooth inner barrel equipped with a cutting shoe, similar to a Shelby tube. The Denison sampler is advanced by rotating the outer core barrel with conventional drill rods, flushing the cuttings from the borehole with fluid (i.e., water or mud), and applying downward pressure on the sampler. Although the outer barrel is rotated to advance the sampler, the inner barrel remains stationary. The Denison tube can also be equipped with a basket type retainer, similar to those used in split-barrel samplers, to aid sample retention in cohesionless strata.

The inner barrel of the Denison sampler is adjustable. It can be set flush with the end of the outer barrel or it can extend up to six inches beyond the end of the outer barrel, although it is general practice that the inner barrel extends no more than three inches beyond the end of the outer barrel. This distance is dependent upon the stratum being sampled. In soft/loose/erodible strata the inner barrel should typically extend up to three inches beyond the end of the outer barrel. The inner barrel is advanced/pushed ahead of the outer core barrel, which is very similar to the standard Shelby tube procedure, while the outer barrel rotates and removes material from around the sampler to make it easier to advance the sampler. In stiffer/denser strata the inner barrel should generally be flush or extend minimally from the end of the outer barrel. The outer core barrel rotates to advance the sampler and cut the sample, and the inner barrel further trims the sample as it enters the inner barrel.

Although the distance the inner barrel extends beyond the outer barrel of a Denison sampler is adjustable, it must be manually set prior to placing the sampler in the borehole. This requires judgment and sometimes a trial and error type strategy to determine what setting/extension results in the best sample recovery. Pre-setting the extension distance is also a disadvantage when drilling in stratigraphy with alternating soft and hard layers. As a result of this disadvantage, the Pitcher sampler was developed.

Unlike the Denison sampler, the Pitcher sampler has a spring loaded inner barrel that automatically adjusts the extension of the inner barrel beyond the end of the outer barrel based on resistance during drilling. In soft stratum, the spring extends so that the inner barrel shoe protrudes into the soil below the outer barrel bit and prevents/reduces disturbance to the sample from the drilling process/fluid. In stiff stratum, the spring compresses until the cutting edge of the inner barrel shoe is flush with the crest of the cutting teeth of the outer barrel bit. Since the Pitcher sampler automatically adjusts to the soil conditions, it is especially recommended for sampling varved soils or stratigraphy with alternating hard and soft strata. Various Pitcher sampler sizes are available. Samplers that obtain sample diameters of three, four and six inches, and lengths of three to five feet are most commonly used.

The Denison and Pitcher tube samplers are not commonly used for Department projects, and neither Publication 222 nor ASTM has specifications for sampling with these samplers. The U.S. Army Corps of Engineers (USCOE) EM-1110-1-1804 does provide guidance for using Denison and Pitcher tube samplers. If these samplers are specified for use on a Department subsurface investigation project, a modification to the standard SBSTC specifications must be made to provide the contractor directions/requirements for performing this work.

3.6.4 Rock Core Drilling and Sampling (Non-destructive Rock Core Drilling)

Bedrock sampling is required for most subsurface explorations, particularly subsurface explorations for structures and roadway rock cuts and for project sites where rock is at a shallow depth. When bedrock sampling is required, rock core drilling must be performed. Rock core sampling is performed to:

- Determine the elevation of the top of bedrock
- Obtain rock core samples for visual inspection and laboratory testing
- Identify the bedrock type
- Describe bedrock color, hardness, weathering, bedding and discontinuity spacing, dip magnitude, and Rock Quality Designation (RQD).

During rock core sampling, subsurface features encountered that could influence the determination of a proper foundation may include voids, mines, or other zones of weakness such as soil or clay infillings, discontinuities, and shear zones/faults. These features are often identified not only in the recovered rock core samples but also during changes in drilling such as:

- Rate of Drilling – Increased or decreased drilling rate, which include tool drops or rapid rates of advancement as these could be indicators of voids, soft zones, clay soil seams, or highly weathered rock. Decreased drilling rates or lack of advancement could indicate very hard rock conditions. Blocking off or plugging of the core barrel can indicate highly fractured rock conditions or presence of clay seams.
- Drill Water Conditions– Changes in the amount of drill water return to the surface or loss of drill water return entirely can indicate the presence of voids, open discontinuities, faults, weathered zones, and soil seams. Drill water return color

should be documented for each core run as this can be an indicator to the degree of rock weathering.

- Rock Core Sample Observations and Handling– If rock core losses are observed documentation should identify and provide an interpretation for the core loss particularly if the losses are thought to represent conditions different from the core recovered. Measurements and observations such as core loss, core recovery, and RQD should be measured while the core remains in the split inner tube, and not after core segments are fitted together and placed in the core box. Rock core samples should be oriented so that bedding or foliation is observed at its maximum dip.

Rock core drilling is the process of recovering cylindrical samples (cores) of bedrock. A core barrel equipped with a drill bit is attached to the bottom of the drill rods, and as the rods and core barrel are rotated and advanced, the core is collected in the core barrel. Drilling fluid, which typically consists of water but can also be a variety of slurries, is pumped through the drilling rods and core barrel during advancement to flush cuttings from the boring and to lubricate and cool the drill bit.

A variety of rock core drilling tools are available. Sizes/diameters of drill casings, rods and core barrels have been standardized by the Diamond Core Drill Manufacturers Association (DCDMA), and letter designations are used to differentiate the various sizes and types. The sizes were selected by the DCDMA so that each tool size can fit inside the next larger size in case telescoping of the hole is needed, for instance when drilling in a difficult formation. A listing of some of the more common sizes available is included in ASTM D 2113. Rock core drilling must be performed in accordance with Publication 222, Section 204 and ASTM D 2113.

3.6.4.1 Core Barrels

The outside diameter of available core barrels typically ranges from approximately 1.2 to 3.9 inches, and the rock core samples that these core barrels obtain range from approximately 0.7 to 3.2 inches in diameter. For Department projects, the minimum outside diameter of the core barrel permitted is 2.98 inches, which results in core sample diameters ranging from approximately 1.75 to 2.3 inches. This minimum diameter corresponds to the NW, NX, and NQ DCDMA core barrel designations. These “N” designation barrels will typically provide acceptable results for most bedrock encountered in Pennsylvania. Larger diameter core barrels (“H” and “P” designations) typically will provide better quality core samples. Therefore, when drilling in a formation that is known to be weak, friable, etc. and rock core recovery and quality may be a problem, consideration should be given to requiring larger core samples.

Core barrels can be single, double, or triple tube. The single tube core barrel is the most basic core barrel, subjects the rock core to the most disturbance, and generally results in poor drill core quality and recovery. The rock core in the single tube barrel is exposed to drilling fluid over its entire length during the coring process and rotates with the core barrel once the core is broken free from the in-place rock. Consequently, rock core recovery and quality are often poor when a single tube core barrel is used. Therefore, single tube barrels are not permitted for use on Department projects. Double tube barrels are the current minimum requirement for Department

projects per Publication 222, Section 204. Double tube barrels better protect the rock core during drilling, which helps improve rock core recovery and quality. The double tube barrel includes an inner steel barrel that protects the core from exposure to drilling fluid, although the bottom of the core may be subject to the drilling fluid during the coring process. A triple tube core barrel is similar to the double tube barrel but it includes an additional liner to better protect the rock core. The triple tube core barrel provides the best core protection, and will yield the best rock core quality and recovery. When drilling in a formation that is known to be weak, highly jointed, friable, etc. and rock core recovery and quality may be a problem, consideration should be given to requiring a triple tube core barrel.

3.6.4.2 Inner Core Barrels

The inner barrel(s) of the double and triple tube systems can be either rigid or swivel type. A rigid type inner barrel is fixed to the outer core barrel and rotates with the outer barrel. This rotation causes problems, including crushing and grinding of the core, and core blockage. Per Publication 222, Section 204, rigid type inner barrels are not permitted on Department projects. Swivel type inner barrels are required on Department projects. Swivel type barrels are the most common type used. They are attached to the outer barrel with a bearing that allows the inner barrel to remain stationary while the outer barrel rotates. Since the inner barrel does not rotate, a much higher level of protection is provided to the rock core.

The inner barrel of the double and triple tube systems can be either solid or split. Rock core is removed from the solid inner barrel by tapping on the barrel with a hammer and catching the core as it falls out of the bottom of the barrel. When the rock is thinly bedded, laminated, or closely fractured, removal of the core from the barrel can be problematic. It is also difficult to “reassemble” the recovered core to its in-situ condition for measuring recovery and RQD, and providing an accurate description. Therefore, per Publication 222, Section 204, split inner barrels are required on Department projects. Split inner barrels are split longitudinally in half, and held together with special tape placed at numerous locations along the barrel. Once a core run is completed and the inner barrel is removed from the outer barrel, the tape is cut and the barrel halves are opened. The core in the inner barrel can be more accurately measured, described and placed into the core box compared to core retrieved from a solid barrel. Use of a split inner tube allows the inspector to examine the rock core in nearly the same position as it was in the subsurface. It is advantageous for the drilling contractor to have two or more split inner barrels on site to allow rock core drilling to proceed while the inspector measures and describes the rock core from the previous core run while still in the split inner barrel. Alternatively, a split/half round PVC pipe or a spare/damaged half of a split inner barrel can be used to transfer the core from the inner barrel being used for rock core drilling.

The double and triple inner barrel systems can be either conventional or wireline. Both types can be used on Department projects, but the wireline system is more common and efficient. Removal of the inner barrel from a conventional core barrel is done by removing the core barrel from the borehole, which requires removing all the drill rods from the borehole after drilling of each core run is complete. Once the core barrel is removed, the outer barrel is disassembled and the inner barrel is removed. If additional coring is required, the core barrel and drill rods must be reinserted into the borehole. This process is tedious and time consuming, and returning the core

barrel to the previous depth can be difficult, particularly in fractured rock formations. The wireline core barrel system allows the inner barrel to be retrieved while the core barrel and drill rods remain in place in the borehole. A tool, referred to as an overshot mechanism, is lowered on a winch cable (i.e., wireline) through the drill rods to the core barrel. The mechanism locks onto the inner barrel, and it is removed from the hole. The overshot mechanism is also used to return the empty inner barrel into the outer core barrel so that rock core drilling may continue. The wireline system is much faster than the conventional system, particularly for deep borings and/or where considerable coring is required.

Conventional (i.e., not wireline) core barrels are available in three series, including “G”, “M” and “T”. The “G” series is the most simple and rugged. These barrels expose the bottom of the core to the drilling fluid which can erode the core. This series is not permitted on Department projects. The “M” series core barrel is the most commonly used and is required on Department projects. This series limits the exposure of the core to drilling fluid which minimizes erosion of the core. Wireline core barrel systems, which are designated by “WL” are very similar to the “M” series conventional core barrel. The “T” series is a thin walled barrel that provides a larger core to borehole size/diameter ratio.

3.6.4.3 Core Bits

A rock core drilling bit is attached to the leading end of the core barrel, and the bit cuts the rock core from the bedrock mass. Numerous types of bits are available, including diamond bits, carbide bits, and sawtooth bits. Carbide and sawtooth bits are made from metal, and are only suitable for softer rocks or very dense overburden. These bits are less expensive than diamond bits but they do not have the longevity of diamond bits, and the drilling rate is slower compared to the rate when using a diamond bit. Diamond type bits provide the best quality rock core and can be used for soft and hard rock. Diamond bits are required for Department projects.

There are numerous types/designs of diamond bits, and the drilling contractor will select the specific diamond bit design to use based on the rock type anticipated for the specific project. Bit selection is dependent upon the diamond type and matrix, rock formation (i.e., hardness and grain size), drill equipment power, and core barrel type. Sometimes the drilling contractor will try various diamond bit types on a project to determine which provides the best quality rock core, cores the most efficiently, and lasts the longest. Diamond bits do eventually become dull and require sharpening/reconditioning. If during rock core drilling the quality of the core deteriorates and/or the coring rate decreases, a likely cause is a worn core bit. The bit should be replaced as necessary.

Diamond bits can either be surface set or impregnated. Surface set bits are used for most conventional drilling operations. Larger diamonds are used for softer rocks, and smaller diamonds are used for harder rocks. These bits can be problematic in formations that have significant variations in rock hardness. Impregnated diamond bits include a mixture of diamonds and metal powder, and typically have a longer life than the conventional diamond bits. These bits can be used in both soft and hard rock formations, and they are typically used on wireline core barrels since bit longevity is crucial. The bit longevity is important to ensure that the drill rods and core barrel do not have to be removed while drilling a boring to replace the bit.

Drilling fluid flow through diamond bits can either be side or face discharge. Discharge through the side (inside to outside) is normally acceptable, especially in harder rocks that are not sensitive to erosion. However, if drilling in softer formations and core recovery is a problem, consideration should be given to the use of a bottom discharge bit to reduce core loss from erosion.

Just above the core bit is the reaming shell. The reaming shell enlarges the hole to the final diameter, and provides adequate clearance to allow drilling fluid and cuttings to flow to the surface. The reaming shell also serves as a centralizer for the core barrel. Similar to the bit, the reaming shell can either be surface set with diamonds or impregnated with diamonds and metal powder. Alternatively, the reaming shell can be made with tungsten carbide strips or other hard surface materials.

3.6.4.4 Concrete Core Drilling and Sampling

For many subsurface exploration projects, it is necessary to obtain representative concrete, bituminous concrete, or other man made materials for pavement design, and to obtain soil and rock samples that underlie these materials. A concrete cutting/coring barrel is used to collect these samples and usually consists of a barrel with a diamond impregnated shoe capable of cutting concrete, including reinforced concrete. Similar to rock coring water is used to cool the bit and clear drill cuttings. Concrete core drilling must be performed in accordance with Publication 222, Section 205 and ASTM C 42.

3.6.5 Unsampld Rock Drilling (Destructive Rock Drilling)

When rock drilling is needed but rock core samples are not required, destructive rock drilling is typically more efficient and cost effective than rock core drilling. Destructive rock drilling is useful for:

- Determining the top of bedrock elevation and bedrock type
- Investigating the variation in the surface of pinnacled or variably weathered bedrock
- Defining voids and soil seams below the top of bedrock in solution prone formations (e.g., limestone, dolomite, marble, etc.)
- Defining limits of voids or seams in coal bearing formations
- Installation of instrumentation (e.g., inclinometers, borehole camera, etc.)
- Selecting pile or drilled shaft tip elevations
- Estimating the need for predrilling for pile installation

Destructive drilling is typically only performed after sufficient rock core borings have been conducted to characterize (e.g., type, hardness, quality, presence of seams/voids, etc.) the bedrock at the project site. Destructive drilling is generally used to augment rock core drilling information, rather than replace rock core drilling. Destructive drilling can be performed during the design phase of a project, but more commonly destructive drilling is used during construction. Destructive drilling is performed using equipment different than that used to

perform rock core drilling. Thus, there will be additional mobilization costs associated with the destructive drilling if rock core drilling is also being conducted. Therefore, when assessing if destructive drilling should be performed to supplement rock core drilling, much consideration must be placed against the greater efficiency of destructive drilling.

Destructive rock drilling can be performed with a variety of equipment, but typically is most efficiently accomplished by the air rotary method. This method is often referred to as “air track drilling” because compressed **air** is used, and the drill equipment is typically mounted on **tracked** machinery. The air rotary method uses air and rotation of the drill rods to excavate/drill the bedrock. A drill bit attached to the drill rods is rotated while compressed air is forced through the drill rods. The compressed air exits through the drill bit to remove cuttings (rock chips and dust) from the hole. Another version of the air rotary method, referred to as a down-the-hole hammer, utilizes a hammer in addition to air and drill rod rotation. The hammer is attached to the drill rods just above the drill bit, and compressed air is used to activate the hammer as well as remove cuttings from the hole. The hammering action of the drill bit, in addition to the rotation of the drill rods, improves the rate of rock excavation, particularly in harder rock. Water is often utilized to suppress rock dust generated from the drilling process and to facilitate cooling of the hammer bit. Drill bit diameters for the air rotary method typically range from approximately one to three inches, whereas down the hole hammer drill bits typically range from 4 to larger than 20 inches.

Although core samples of the bedrock are not obtained from the air rotary method, considerable information can be obtained, particularly when the drilling is observed by a trained individual. The top of bedrock is generally easily determined because the drill reacts differently when advanced through soil (i.e., unconsolidated material) compared to bedrock (i.e., consolidated material). Also, the cuttings, which consist of rock chips and dust, blown from the hole can be examined by a trained individual to determine the bedrock type. Often a fine screen or sieve is used to collect and examine the cuttings. Changes in the bedrock, for instance weathered intervals or soft seams, can often be identified by color variations in the rock cuttings and/or a change in the behavior of the drill rig. The rate of rod advance, per foot, should be observed and recorded. This advancement rate can provide an indication of relative bedrock hardness, and the presence of voids and soil seams.

Caution must be used when using the air rotary method in karst topography (e.g., limestone, dolomite, marble, etc.). Due to the high volume of air injected into the borehole, the air rotary method can deteriorate subsurface conditions, particularly if very loose soils and/or voids are present. If groundwater is encountered, these problems can be exacerbated since the compressed air forces the water out of the borehole and throughout the subsurface, which can cause additional erosion, formation of voids, etc.

Air rotary drilling is primarily intended for rock drilling (i.e., consolidated materials). Air rotary drilling has no or very limited use for sampling, identifying or describing soil deposits. Air rotary can advance through soil, typically for the sole purpose of reaching and drilling bedrock. In some soils, like wet, clayey soil, the soil may clog the drill bit and prevent the soil cuttings from being blown to the surface. If this occurs, boring advancement can be difficult, and may not be possible. If advancement through soil by air rotary drilling is not possible, soil drilling

must be performed using different drilling techniques/equipment. Temporary casing can be installed to support the overburden and permit air rotary drilling of rock.

3.7 TEST BORING PROCEDURES/CONSIDERATIONS

3.7.1 Continuous versus Interval Standard Penetration Testing

Split-barrel sampling can either be done continuously or at intervals. Continuous split-barrel sampling, which is typically used for Department projects, means sampling is performed every 1.5 feet without any gap/space between the bottom and top of consecutive samples. Interval sampling means there is a gap/space between the bottom and top of consecutive samples. The typical intervals are three feet and five feet, and this distance is measured from the top of consecutive samples. Therefore, three feet interval sampling means split-barrel sampling starts every 3 feet, and there is a 1.5-foot gap/space between the bottom and top of consecutive samples. The Publication 222 SBST Contract includes pay items for continuous and interval sampling, a premium for borings on water, and premiums for deep (60 – 120 ft.) and extra deep (>120 ft.) sampling.

Continuous sampling provides a more complete indication of the subsurface conditions compared to interval sampling, but is more time consuming and will cost more than interval sampling. Test borings performed for structure foundation design, landslide explorations, or where the overburden is relatively thin typically should utilize continuous sampling. Interval sampling should typically be used for geotechnical roadway design where the overburden is relatively thick and subsurface conditions are uniform. In thick overburden deposits, consideration can be given to performing a combination of continuous and interval sampling in the borings. Continuous sampling can be used in the upper portion of the borings where stresses from foundations and embankments are generally higher and a more detailed understanding of the subsurface conditions is needed. Interval sampling can be used at lower depths in the borings where stresses from proposed construction will be lower, and therefore, a detailed understanding of the subsurface is not as critical.

3.7.2 Standard Penetration Test (SPT) Refusal

Generally speaking, SPT refusal occurs when very hard/dense material is encountered, and the split-barrel sampler cannot be advanced or can only be advanced with excessive hammer blows. Various definitions of SPT refusal exist throughout the drilling industry. Publication 222, Section 202 must be followed for Department projects, and defines SPT refusal as one of the following:

1. 50 blows/drops of the hammer advances the sampler one tenth of a foot (0.1 ft.) or less
2. 50 blows/drops of the hammer advances the sampler three tenths of a foot (0.3 ft.) or less for two consecutive 6-inch sampling intervals.

Once SPT refusal is achieved, the test boring will either be terminated or rock core drilling will commence. Whether to terminate the boring or begin rock core drilling will be

based on the purpose of the test boring, and must be indicated in the Schedule of Proposed Borings. It is important to follow Publication 222, Section 202 refusal criteria guidelines to assure consistency on individual projects across the state, and so that drilling contractors can bid the work accordingly. Even if rock core drilling is performed, it is possible that SPT will be required in the test boring if rock core recovery is less than 20 percent. See Publication 222, Section 202 for detailed guidance on advancing the test boring after SPT refusal is encountered, and see [Section 3.6.4](#) for discussion on rock core drilling.

3.7.3 Rock Core Drilling Core Run Lengths

As discussed above and as indicated in Publication 222, Section 202, if required/necessary, rock core drilling commences once SPT refusal is achieved. Rock core drilling is typically performed by coring in intervals (i.e., core runs) ranging in length from two to five feet. Five feet is the maximum core run length permitted on Department projects. The core run length used is based on several factors, but must be in accordance with Publication 222, Sections 202 and 204.

The first rock core run should not exceed two feet in length. Limiting the first core run to two feet aids in assessing if the quality of the material is suitable for continued rock core drilling, or if a cobble or boulder was encountered. Based upon the material and material quality encountered in the first 2-foot core run, a determination can be made as to whether SPT or rock core drilling should continue. Additionally, the top few feet of bedrock is typically of poorer quality (i.e., weathered, softer) than the bedrock below; therefore, a limited core run length of two feet is required to better define the condition of the bedrock near the surface. If a longer core run, say five feet, is used at the start of rock core drilling and poor rock core recovery is obtained, it is often difficult to determine if the poor recovery is only from the upper portion of the 5-foot core run, or if it is consistently poor throughout the entire core run. Therefore, limiting the length of the first core run allows more accurate characterization of the bedrock, permits better assessment of acceptable rock bearing elevation for spread footings and deep foundations, and helps estimate means for excavating bedrock.

The rock core recovery percentage (i.e., length of core recovered divided by total length of core run) from the first 2-foot core run is used to determine the next drilling action. If the rock core recovery percentage is 80 percent or higher, a core run of up to five feet in length can be attempted. However, if the first 2-foot core run recovery is less than 80 percent but greater than or equal to 20 percent, the next core run must be limited to no more than three feet in order to help further define the transition between the poorer and better quality bedrock. If the first 2-foot rock core recovery percentage is less than 20 percent, additional standard penetration testing may be needed prior to resuming rock core drilling. Refer to Publication 222, Section 202 for more details.

3.7.4 Sampling Highly Weathered/Very Soft Bedrock

Thick zones of highly weathered bedrock, which can be common with schist and gneiss, and very soft bedrock, which can be common with claystone, coal and shale, are often very challenging to sample. Such highly weathered and/or soft bedrock is typically hard enough to

result in SPT refusal, so a sample cannot be obtained with a split-barrel sampler. However, this material is frequently weathered/soft enough that drilling disturbance and water used during rock core drilling destroys most, if not all, of the sample. Extra care must be taken when sampling these types of bedrock so that adequate sample can be obtained for classification/characterization purposes. Use the requirements of Publication 222, Section 204 to achieve the best results possible.

Avoid conducting long lengths of SPT or rock core drilling if no to minimal sample is recovered. Even if SPT refusal criteria per Publication 222, Section 202 is not quite achieved, but little to no sample is recovered after several attempts (e.g., 3 sample intervals or 4.5 feet), consider a short (e.g., 2 to 3 feet) rock core run. When rock core drilling in these types of bedrock, discuss drilling techniques/methods with the drill operator to help improve sample recovery. Discuss types of drill bits available, amount of water delivered to the bit during coring, and down pressure/rate of advancement of the drill rods/core barrel. Use 2-foot core runs, and if no or minimal sample is recovered after two or three core runs, attempt sampling by SPT. Standard penetration testing, even if no to little sample is recovered, provides more information (i.e., penetration resistance) for design and construction considerations compared to core runs that obtain no to little recovery.

3.7.5 Sampling Pinnacled and/or Voided Bedrock

Rock core drilling in pinnacled and or voided bedrock can be difficult and challenging. Pinnacled bedrock (i.e., carbonate rock with highly differential weathering) can have extreme variation in the top of bedrock surface over very short distances. Drilling tools, including augers, casing, split-barrel samplers and core barrels tend to “wander” on the pinnacled surface. Note that casing seems to perform better in pinnacled rock compared to hollow stem augers. This wandering can cause augers/casing to bow/bend to the point of breaking, or enough to prevent sampling tools from reaching the bottom of the boring. Split-barrel samplers often get wedged in the fractures in the pinnacled surface or simply shear off at the connection to the sampler rods. Due to this drill tool “wandering”, boreholes sometimes must be abandoned and a new test boring performed nearby.

Carbonate bedrock (e.g., limestone, dolomite, marble) tends to have more soil seams than other bedrock formations, and these seams can range in thickness from less than an inch to several feet. Carbonate bedrock also can have rock “ledges” that overlie thick soil stratum, and wide vertical or near vertical soil filled seams. Consequently, soil sampling is often required in carbonate bedrock after bedrock core drilling has begun. The purpose of the boring and the information required for design and construction will determine whether sampling of soil seams below the top of bedrock is required. If soil sampling is needed below the top of bedrock, a few options are available as discussed below.

For relatively thin soil seams, say less than a few feet in thickness, a small diameter split-barrel soil sampler (i.e., 1.5 to 1.75 inch OD) may be appropriate to use. A standard 2-inch OD split-barrel sampler is too big to be inserted through a standard N-size wireline core barrel. These core barrels typically have a 2-inch or smaller inside diameter. This small diameter split-barrel soil sampler is advantageous because the core rods and barrel do not have to be removed

from the hole to use this sampler, and the core rods function as casing to support the borehole while using the small diameter sampler. This sampler does not meet the Publication 222, Section 202 or ASTM specifications for normal SPT sampling, however, this sampler can be considered for use in such special circumstances, as when attempting sampling and penetration testing below top of bedrock surface, and available drill casing does not permit passage of a standard split-barrel sampler. Note that, due to the large variation in available core drill barrel diameters, and since some drilling contractors use customized tools, it is possible that a standard split-barrel soil sampler will fit through some N-size core barrels.

Sampling of thick soil deposits below the top of bedrock will require that the core drill rods and core barrel be removed from the borehole if a standard 2-inch OD split-barrel soil sampler cannot be inserted through the core barrel. Assuming the core drilling rods have to be removed to insert the standard split barrel soil sampler, the borehole will be partially unsupported. In these cases, it is necessary to advance the augers/casing through the bedrock. As discussed earlier, augers most likely will not be able to advance past the top of bedrock. Consequently, drilled in casing equipped with a diamond bit cutting shoe will most likely be necessary to case the hole through bedrock and into the soil seam. Since casing may need to be advanced through bedrock when drilling in carbonate bedrock, and since casing often performs better when pinnacled bedrock is encountered, the use of drilled casing should be considered (and may be required) when performing test borings in carbonate bedrock.

Voids in the bedrock, either occurring naturally in carbonate rocks or manmade from mining, often present challenges or problems when rock core drilling. When a void is encountered, drill water return to the surface will be lost. Loss of drill water return can also occur in bedrock that does not have voids, for instance when an open fracture or highly weathered/broken zone of bedrock is encountered. Nonetheless, drill water return is beneficial to the driller to help ensure that adequate water is being delivered to the bit for cooling/lubrication and to flush cuttings, so losing drill water return is a disadvantage. Encountering voids can also present a problem if the core drilling rods and barrel have to be removed from the borehole (e.g., to replace the bit, free a stuck inner barrel, or other mechanical issues), because it can be difficult to replace the core barrel into the borehole beneath the void. The use of wireline drilling tools significantly reduces the likelihood of having to remove the rods from the borehole, but it is still occasionally necessary.

3.7.6 Inclined Drilling

Vertical borings are typically sufficient for exploration in horizontal to near horizontal bedded geology and are used on the majority of subsurface exploration projects. However, for steeply bedded rock, or where complex folded and faulted geology is present, inclined borings may be necessary to sufficiently explore subsurface conditions and increase the likelihood of the boring intersecting steeply inclined discontinuities. In addition, inclined borings may be used where surface obstructions prevent vertical holes.

The location and spacing of borings are dependent upon the orientation of the geologic structure. For example, if known or suspected near-vertical joints, shear zones, or faults need to be identified for design purposes, inclined borings may be completed to characterize them. The length of the inclined boring should be based on the anticipated foundation type and lithology. Inclined borings are more time intensive and costly to obtain than vertical borings, and not all drilling rigs are capable of completing inclined borings. SPT sampling of the overburden is not feasible in an inclined boring. Boring inclinations up to 30° from vertical are usually obtainable. Inclinations greater than 30 degrees are generally not possible. It is important to understand that inclined borings are subjected to alignment deviations due to the natural tendency of angled drill tooling to deflect downward toward a vertical orientation with depth. Causes of borehole deviation can include the use of worn or damaged equipment, hard rock zones, voids, and discontinuities within the rock.

3.7.7 Grouting/Backfilling Boreholes

Once information is obtained from a test boring, it must be grouted/backfilled. Boreholes must be backfilled with cement grout for a variety of reasons, including safety, preventing groundwater/aquifer contamination, and preventing ground subsidence. Publication 222, Section 210 provides requirements/guidance for backfilling of boreholes.

There are two primary reasons for properly backfilling boreholes. The first is an environmental requirement to prevent potential contamination or cross-contamination of aquifers. Boreholes must be properly backfilled to prevent surface water from entering and possibly contaminating the groundwater. Possible groundwater contamination from surface water influence is a particular concern when the borehole is in the vicinity of a spring or well that is used as a drinking water source. When boreholes encounter multiple voids, the borehole must be grouted in stages to ensure that the borehole is backfilled below and above each void so that regional groundwater flow and flow within mine workings are not altered. Grouting boreholes that have encountered multiple voids or a “large” void will require the use of grout baskets/plugs to prevent grout loss into the voids. Small voids in carbonate bedrock can be grouted along with the borehole in lieu of using a basket/plug.

The second main reason for backfilling boreholes is for safety. Borings that are not backfilled, or not backfilled properly, can seriously injure a pedestrian. Cases have occurred where pedestrians have been injured from stepping into an open borehole, and they have requested financial compensation from the Department and/or consultant. Therefore, backfilling boreholes must not only be done to protect the public, but it must be done to avoid potential costly disputes. Boreholes not backfilled or improperly backfilled can also result in ground

settlement that could result in harm to the traveling public or damage to public and/or private property. Open or improperly backfilled boreholes in carbonate bedrock can be conduits for surface waters to enter the subsurface and cause erosion that eventually leads to ground subsidence at the surface. The cost to backfill a borehole is minimal compared to the potential cost of an injury to the public or damage to property.

Another potential critical need to properly backfill boreholes is when artesian conditions exist. Boreholes not properly grouted in artesian conditions often result in latent damage to the subsurface in the surrounding area which can undermine existing structures or cause subsurface damage that is costly to repair or remediate. Borings drilled into an artesian aquifer that produce flowing water at the ground surface must be backfilled with extreme care. Improper backfilling when these conditions are encountered could allow leakage/piping that may degrade the subsurface and cause subsidence at the ground surface. If water flows from the borehole to the ground surface, augers/casing should be extended above the ground surface to contain the water (i.e., stop the flow). Once the flow is stopped, grouting should commence in accordance with Section 210. Augers/casing will have to temporarily remain in place during grouting to prevent any flow of water from the borehole. If auger/casing stick up above ground surface is not adequate to stop the flow from an artesian aquifer, additional measures will be required to properly backfill the borehole. These measures could include the use of mechanical packers and/or heavy weight grout.

Publication 222, Section 210 provides grout mixes and procedures for backfilling boreholes. Three grout mixes are provided in Section 210. Type 1 is a neat cement grout that should be used for backfilling most boreholes. Type 2 and 3 mixes contain bentonite and are intended to backfill boreholes that are drilled in soft soils and soft/porous bedrock where grout loss may occur and for backfilling around inclinometer casing. As indicated in Section 210, grout should be placed in the borehole by pumping through a grout pipe placed at the bottom of the borehole. The pipe used to monitor groundwater levels should be used to place the grout so that the borehole remains cased/supported until grouting is completed. Similar to placing concrete under water, the grout must be deposited through a pipe placed at the bottom of the borehole so that any water in the borehole “rides” on top of the grout to the surface and does not get trapped in the borehole. Water trapped beneath grout will often lead to settlement of the grout and subsidence at the ground surface.

3.7.8 Groundwater

It is necessary to understand the groundwater conditions on a project site in order to properly design the proposed highway facilities and determine the impacts of groundwater during construction. Groundwater levels must be observed and recorded during the subsurface exploration, and for some situations long term observation/monitoring is necessary. With a thorough understanding of the groundwater conditions, necessary mitigation measures and associated costs can be planned and implemented

3.7.8.1 Groundwater Monitoring during Subsurface Exploration

Groundwater must be observed and recorded during drilling of test borings, and for a minimum 24-hour period after the completion of drilling prior to grouting. Where groundwater is located within a permeable soil stratum, like sand or gravel, it can often be detected during split-barrel sampling because the soil samples, and sometimes even the split-barrel sampler, will be visibly wet once groundwater is encountered. When groundwater is located within a less permeable soil stratum, like clay and clayey silt, groundwater is difficult to detect during sampling. Requirements for observing/measuring groundwater levels in test borings are included in Publication 222, Section 209.

Since the groundwater elevation is often difficult to determine during drilling, it must be monitored after the completion of a boring. At a minimum, zero hour and 24-hour groundwater levels must be observed in all borings. After the completion of drilling, but before the augers/casing/mud are removed, PVC pipe with a slotted or perforated section at the bottom must be inserted into the borehole. This PVC pipe will prevent the boring from completely collapsing once the augers/casing are withdrawn and permit groundwater observations.

The zero-hour groundwater reading is taken immediately after the augers/casing are removed from the boring. This water level typically does not represent the static groundwater level. If water is introduced into the boring during drilling for rock core drilling or flushing of the borehole, the zero-hour water level in the boring will often be higher than the static groundwater level. Conversely, when borings are drilled in low permeability materials and water is not introduced into the boring during drilling, the zero-hour level is typically lower than the static groundwater level because it takes time for groundwater to infiltrate into the borehole and stabilize.

The 24-hour groundwater reading is taken approximately 24 hours after the zero-hour reading. This generally provides a better representation of the static groundwater level since the water level in the borehole has time to equalize with the water level in the surrounding soil/rock. When borings are left open (i.e., not grouted) for a period longer than 24 hours, additional groundwater readings should be taken and recorded on the boring log. At a minimum, the water level in the boring should be recorded at the completion of drilling, 24 hours after the completion of drilling, and immediately prior to grouting the boring. When observing the groundwater elevation in very clayey soils, longer periods of observation (possibly including installation of a piezometer) may be necessary, since it can take significantly longer than 24 hours for water levels to stabilize in these very low permeability soils.

During the monitoring period ensure that surface water does not enter the boring since this will most likely elevate the water level in the boring. This can usually be accomplished by mounding soil around the top of the boring. Trapped drill water may also remain in the boring 24 hours or more after the completion of drilling. If trapped drill water is suspected it can be removed with a mechanical bailer or by other means, and the water level can be monitored for an additional 24 hours or more. Be skeptical of groundwater levels that are measured at or above soil sample elevations that are described as dry or damp (or moist for granular samples). When

drilling near creeks, streams, rivers, etc., expect the groundwater level to be at or near the normal water surface elevation of the waterway.

Per Publication 222, Section 209, water levels must be measured with electronic water level indicators. Care must be taken when using these indicators. Prior to inserting into the borehole, submerge the water level indicator probe into a bucket of water to ensure it is operating properly. Additionally, these water level indicators, and in particular, the tape type, can adhere to the side of the PVC casing due to water adhesion, and signal a higher water level than actually exists. If this is suspected, use an alternate method, such as a weighted tape, in conjunction with the electronic indicator to confirm the actual water level.

3.7.8.2 Long-Term Groundwater Monitoring

For projects where long-term groundwater readings are needed to determine if there are seasonal or precipitation related variations in the groundwater level, piezometers should be installed in designated test borings. There are numerous types of piezometers, including standpipe/Cassagrande, vibrating wire, and pneumatic. Publication 222, Section 206 provides a specification for the installation of standpipe piezometers. Long-term groundwater monitoring should be considered for landslide remediation projects, roadway projects with “large” proposed cuts/fills, and other projects where groundwater could significantly impact design or construction.

3.7.8.3 Confined and Unconfined Aquifers

Groundwater occurs in aquifers under unconfined or confined conditions. Multiple aquifers at various depths typically underlie a project site, although usually only the uppermost aquifer is encountered during a subsurface exploration for roadway projects because the borings are typically shallow with respect to the depth of many aquifers.

Typically, the shallowest aquifer at a project site is an unconfined aquifer. An unconfined aquifer is open to receive water from the ground surface, and the aquifer water level, often referred to as the groundwater table, is free to fluctuate up and down. During wetter seasonal periods (i.e., winter and spring) the water level of the aquifer typically rises, and during dry seasonal periods (i.e., summer) the water level typically falls. In Pennsylvania, unconfined aquifers often recharge streams and rivers, and therefore the unconfined aquifer water level is very similar to the level of the adjacent stream/river. Sometimes “perched” groundwater is located beneath a project site. Perched groundwater is water that is trapped above a low permeability layer, such as clay. This term is used to describe a localized area of groundwater that has a higher level than the level of the larger, regional aquifer.

Where water completely fills an aquifer that is overlain by a relatively impermeable or impermeable layer of soil or rock (i.e., confining layer), the water in the aquifer is said to be confined. A relatively impermeable confining layer is referred to as an aquitard, and a completely impermeable confining layer is referred to as an aquiclude. This confining layer prevents water from above entering into the aquifer. Instead, the aquifer is recharged laterally. These aquifers are often located beneath an unconfined aquifer.

3.7.8.4 Artesian Aquifers

Occasionally, water within confined aquifers is under pressure. These are referred to as artesian aquifers. Artesian aquifers typically occur in valleys or at the base of mountains where confined aquifers are recharged from a connection to water at higher elevations. The head of water from the higher elevations pressurizes the water in the lower confined aquifer since the water cannot rise due to the overlying confining layer. If the overlying confining layer of an artesian aquifer is penetrated by a test boring, the water level in the test boring will rise above the level in the aquifer to a level equal to the water pressure in the aquifer (i.e., height of water level above top of aquifer equals the water pressure within the aquifer divided by the unit weight of water). A well installed into an artesian aquifer is referred to as an artesian well, and if the water in the well naturally flows to the ground surface it is referred to as a flowing artesian well. Artesian conditions can also be encountered when drilling at the base or downstream of a dam/water impoundment.

Extreme care must be taken when an artesian aquifer is encountered by a test boring, particularly if the water flows to the surface. If the flowing water is not stopped, it could carry finer soil particles (i.e., silts and fine sands) to the surface, which is often termed piping. This piping can cause ground settlement due to the loss of the material from underground. If water flows from the top of the augers/casing during drilling, additional sections should be added above ground surface to contain the aquifer water within the augers/casing. Typically, a 5-foot section of casing will be adequate to contain the flow. If the water can be contained within a reasonable height above ground surface, the boring may be advanced to completion. If the water cannot be contained the boring should be abandoned and immediately grouted. Upon completion of the boring, it is critical that the boring is properly grouted to prevent water leakage from the borehole. Note that where flowing water conditions are encountered, foundations which penetrate the artesian aquifer should be avoided where possible, due to potential degradation of the subsurface from piping.

3.7.9 Running Sands

Sometimes when advancing a boring through soil, a condition referred to as “running sands” or “blow in” is encountered. Running sands refer to cohesionless soil, typically sands, which enter the boring through the bottom of the augers or casing. When encountered, the running sand will often fill the casing with several feet or more of soil, preventing the sampler from reaching the bottom of the hole.

This condition occurs when the groundwater level outside of the casing is higher than the water level inside. This difference in water level causes the water pressure outside the casing to force the cohesionless soil into the bottom of the casing. This typically happens when a boring is being advanced through low permeability soil and then a high permeability layer (i.e., sand) is encountered. The low permeability soil does not allow groundwater to flow quickly enough inside of the augers/casing and creates the unbalanced water level. Running sands also occur when fairly watertight casing is used to advance the boring and groundwater cannot easily enter the casing. Removal of the center bit from the casing once the desired sampling depth is reached sometimes creates a vacuum that further accelerates the development of running sands.

Running sands can typically be avoided by introducing water into the augers or casing so that the water level inside the casing is at or above the groundwater level. By doing this, the water pressure at the bottom of the boring will be equal to or greater than the water pressure outside of the boring, and soil will not run into the boring. If running sands are encountered, the boring must first be cleaned by flushing with water using a roller bit, chopping bit, etc. to the bottom of the augers/casing (i.e., the sampling depth). While removing the drill rods from the boring, water should be pumped into the top of the augers/casing keeping them full. The sampler should be lowered to the bottom of the boring, and drill rod stick up above ground surface should be measured to ensure that the sampler is at the proper location. Prior to and during removal of the sampler from the boring, the casing should again be pumped full of water. This procedure should be followed for the remainder of the boring, or until the augers are seated into a less pervious, cohesive material. If filling the casing with water does not prevent running sands, then a heavier drilling fluid, like drilling mud/slurry, must be used to continue the boring.

3.7.10 Pavement Design Borings

Recommended location and depth requirements for borings drilled to obtain information on subgrade material for pavement design are provided in Tables [3.2.4-1](#) and [3.2.4-2](#). One purpose of pavement design borings is to obtain bulk soil samples for laboratory testing to estimate the subgrade strength. The Department's current pavement design procedures require the determination of the California Bearing Ratio (CBR) of the subgrade soil to estimate the resilient modulus (M_r) of the subgrade soil. Another purpose of pavement design borings is to obtain cores of the existing pavement for projects where overlay and/or match-in-kind pavement construction is anticipated.

Soil samples for CBR testing should be collected on all projects where pavement design is required. In addition, the use of dynamic cone penetrometer (DCP) or falling weight deflectometer (FWD) may be considered for estimating resilient modulus if reliable correlations are available and approved by the Department Central Office Pavement Design and Central Office Geotechnical Sections. When collecting samples for CBR, or if conducting DCP or FWD, additional samples should be collected to run index tests including gradation, Atterberg limits, hydrometer, moisture density, unconfined compression tests, and resilient modulus tests so that information and data can be collected to support the Department's initiative to develop a reliable method of correlation to estimate resilient modulus. These additional samples should be sent into the Construction and Materials Division Laboratory for testing. When sending samples please indicate that they are for the purpose of developing a resilient modulus correlation for the Department; however, if they are also for project design, please indicate that as well.

Where new pavement will match existing pavement section(s), pavement design is not needed; therefore, CBR testing is not needed.

Soil samples for CBR testing and pavement cores should be collected at 500- to 1,000-foot intervals along the proposed alignment depending upon the variability of the subgrade. Tighter (i.e., 500 feet) spacing should be used where the subgrade soils vary along the alignment, and wider spacing is appropriate for more uniform subgrades. Ideally, collect CBR samples

within the limits of the travel lanes (i.e., pavement box), and alternate the sampling locations to both sides of the roadway, particularly for multiple lane and divided highways. For existing roadway alignments, it will likely be necessary to core through the existing pavement to obtain the samples. Coring the existing pavement is also beneficial to determine or verify the existing pavement section. For structure replacement projects where match-in-kind pavement is anticipated, obtain at least one pavement core. Ensure the pavement core is obtained beyond the limits of the bridge approach slab. When sampling beneath existing pavement, be sure to penetrate through all existing subbase material before collecting the CBR sample to avoid mixing the subbase with the subgrade.

In some cases, it may be necessary to locate the borings for CBR samples outside the existing travel lanes in order to avoid the need for traffic control; however, this may not be appropriate because the subgrade materials outside of the travel lanes may be different than the subgrade under the pavement. Additionally, the subgrade beneath the pavement may have been modified or stabilized prior to construction. When consideration is given to collecting CBR soil samples outside of the existing travel lanes, study available historical information, including RMS data, design drawings, and as-built drawings, etc. to determine if subgrade stabilization was performed. If a determination cannot be made, it is best practice to collect the CBR samples from beneath the existing pavement section.

Collect soil samples within the top three feet of the proposed subgrade elevation for pavement sections located at-grade or in soil cut. SPT is not necessary for pavement design borings. Additionally, in cuts, collect samples of material above the subgrade if this material may be used to construct roadway embankments. If embankments are to be constructed using known off-site borrow material, collect and test samples from the proposed borrow source. If the location(s) of the borrow source(s) is not known, consider adding requirements in the project special provisions that require the contractor to sample and test proposed material to be used to construct the upper three feet of embankments. In the special provision, provide the minimum required CBR value, consistent with the approved pavement design.

Seventy-five (75) pounds of material is needed to perform the CBR test. However, if the soil sample has material larger than the 3/4 in. sieve, obtain an additional sample so that the oversize material can be replaced with finer grained material per the test method.

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CHAPTER 4 – LABORATORY TESTING

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4.1 INTRODUCTION

This chapter provides minimum requirements, guidelines, recommendations, and considerations for planning laboratory testing programs. This chapter also provides guidance and considerations for selecting soil and rock samples for laboratory testing, and discusses requirements for presenting the laboratory test results. A summary of the most commonly used geotechnical tests is also provided, and detailed recommendations and guidance are provided for some of the more complex tests.

Proper preparation and execution of a laboratory testing program is a critical and necessary part of the geotechnical design process. It will provide soil and/or rock properties needed to perform geotechnical analyses and develop appropriate geotechnical recommendations. It must be understood that all projects are unique, and similar to the subsurface exploration, the laboratory testing program must be carefully analyzed and planned for the specific needs of the project. Simply using the minimum requirements included in this section is not adequate for most situations. For other than very simple and highly favorable conditions, such an approach will likely result in an inadequate design due to insufficient information.

4.1.1 Purpose

A laboratory testing program is performed to supplement visual observations made during the subsurface exploration. A properly planned and executed laboratory testing program will more definitively estimate the engineering characteristics of the soil and rock, and help predict how the soil and rock will behave during construction and the service life of the roadway, structure, etc. The value of adequate laboratory testing cannot be underestimated or overlooked. An appropriate level of laboratory testing provides several substantial benefits, especially relative to the minimal cost of the testing, such as follows:

- Drastically increases the level of knowledge and understanding of subsurface conditions.
- Provides a significantly higher level of confidence in proposed designs.
- Reduces the risk of construction problems, delays, claims, and/or failures.
- Reduces the risk of potential failures and service interruptions post construction.
- Substantially increases the potential quality and longevity of the finished product.
- Has the potential to significantly reduce construction and maintenance costs.

The laboratory test results, along with the information obtained from the subsurface exploration, are needed to select the soil and rock parameters used for geotechnical analyses and design. Relying solely on field observations/descriptions and Standard Penetration Testing (SPT) is not sufficient for most projects. Field descriptions of soil and rock samples are only approximations, and therefore will not always be accurate. Also, SPT can only be used for some types of granular soil, and only provides an estimate of density/consistency. SPT does not directly measure shear strength, can only be used to correlate strength and density from published information, and is only applicable for particular soil types. Furthermore, engineering

properties such as consolidation and shear strength can only be determined from laboratory testing, particularly for fine-grained soils.

The appropriate laboratory testing data provides a higher level of confidence in the soil and rock parameters used for geotechnical analyses compared to only using field observations. This confidence better ensures that the geotechnical design recommendations are appropriate and safe for the proposed construction, while not being overly conservative and unnecessarily costly. The time and cost to perform laboratory testing during the design phase is significantly less than the time and cost associated with a problem during or after construction due to a design based on erroneous soil and/or rock information. Additionally, the use of data from a laboratory test is far more superior, and will provide much more reliable results, compared to situations where back analysis is used to determine soil and rock parameters or provide designs in lieu of laboratory testing. Back analysis is a tool that has limited applicability, must be used in an appropriate manner consistent with actual conditions, and is best used (where appropriate) as a supplemental check to help confirm a design developed using the most suitable level of laboratory testing.

4.1.2 Reasons Laboratory Testing Is Performed

Laboratory testing of soil and rock samples collected during the subsurface exploration is performed for several reasons. The following sections discuss these reasons in detail.

4.1.2.1 Verification of Field Soil Descriptions

Subsurface exploration (drilling) observation and preparation of Inspector's Field Logs must be performed by Certified Drilling Inspectors per Publication 222, Chapter 3.1. The preparation of Inspector's Field Logs requires the Certified Drilling Inspector to visually describe soil samples obtained from the borings. Although the Certified Drilling Inspector must be proficient at describing soil samples, it is important to verify some of these field descriptions with laboratory testing. Laboratory classification testing provides a level of confidence in the Inspector's Field Log. When field descriptions generally agree with laboratory classification testing, field descriptions can typically be used with a high level of confidence. Conversely, if field descriptions and laboratory classification testing do not agree reasonably well, then there is reason for concern and results in a low level of confidence in the Inspector's Field Log. If field descriptions and laboratory classification test results vary significantly, all samples collected during the subsurface exploration must be reviewed by the PGM who must make revisions as appropriate. Note the distinction between the terms **description** and **classification**. A soil description is based on visual observations and simple field tests (e.g., rolling soil threads, squeezing/crushing dried soil balls with fingers, etc.), whereas a classification is determined from laboratory testing (e.g., gradation, Atterberg limits, etc.).

Note that it is important to provide the laboratory classification results to the drilling inspector who prepared the Inspector's Field Logs. Comparing field descriptions to laboratory classifications is an excellent way for drilling inspectors to improve their field description skills/abilities.

4.1.2.2 Determination of Material Properties

Field descriptions of soil and rock samples provide only a general indication of material properties. For example, field descriptions can:

- Distinguish moisture content as being dry, damp, moist or wet.
- Roughly estimate the percentage of gravel, sand, and fine-grained (silt and clay) particles.
- Predict whether a soil sample is non-plastic or has low, medium, or high plastic fines.
- Roughly estimate soil relative density based on SPT-N60 or consistency based on pocket penetrometer and Torvane field tests.
- Qualitatively estimate rock hardness and degree of weathering.

Although these general material property descriptions are useful, it is beneficial and necessary to obtain specific material property values for some of the soil and rock samples. Specific material property values can only be obtained by laboratory testing. These specific values not only verify field descriptions, but they help to establish soil parameters and more accurately perform geotechnical analyses.

4.1.2.3 Estimation of Engineering Properties

Engineering properties of soil (such as consolidation and shear strength) and of rock (such as unconfined compressive strength) are sometimes needed to perform geotechnical analyses. Evaluate and assign these properties as accurately as possible because these values will have a significant impact on the geotechnical recommendations. These properties cannot be estimated with an appropriate level of confidence without the use of laboratory testing, particularly when working with fine-grained soils.

4.1.2.4 Comparison with In-Situ Testing

In-situ testing, such as a Cone Penetrometer Test (CPT), vane shear, or pressuremeter tests, may be performed during the subsurface exploration to estimate engineering properties of the soil. Although in-situ testing can be a very effective technique to estimate engineering properties, it must be performed carefully and correctly to obtain representative results. However, even when in-situ testing is performed correctly, circumstances may exist that yield erroneous results not representative of the actual conditions. Therefore, if in-situ testing is used to estimate engineering properties, laboratory testing should typically be performed to compare the results with the in-situ test results. Note that SPT is an in-situ test; however, this test only provides an indication of relative density/consistency and does not directly measure density, strength, etc.

4.1.2.5 Chemical Analysis

With the exception of field resistivity testing, chemical analyses of soil and bedrock can only be performed in the laboratory. Soil pH, resistivity, sulfate and chloride testing is typically

always needed, especially if deep foundations are proposed to support a structure, and should be considered when there is a potential for deterioration (e.g., corrosion of metals, disintegration of concrete, etc.) when shallow foundations are proposed. These tests can and should be performed in the laboratory. Acid bearing potential of bedrock, and in some cases soil, is needed for projects where acid producing materials may be exposed in cuts/excavations. Chemical analyses of water should also be performed for structures that convey flow from rivers, streams, creeks, etc.

4.1.2.6 Selecting Resistance/Safety Factors

Laboratory shear strength testing of soil may influence the geotechnical resistance/safety factors used for design. For example, the resistance factor used for spread footing foundations bearing on soil is dependent upon whether the soil shear strength parameters used in the analyses are based on laboratory test data or presumptive/assumed values. If laboratory shear strength test data are used, a higher resistance factor is permitted compared to the resistance factor used with presumptive/assumed shear strength parameters. Similarly, when slope stability analyses are performed, shear strength parameters based on laboratory test results typically allow for a lower safety factor compared to when presumptive strength values are used. Consequently, higher resistance factors and lower safety factors associated with laboratory testing often result in more economical designs compared to when laboratory testing is not completed. Refer to the appropriate sections of this publication, or DM-4, as appropriate, for additional guidance.

4.2 DEVELOPMENT OF A LABORATORY PROGRAM

Prior to planning the laboratory testing program, several tasks discussed in [Chapter 2](#) of this publication must be completed. These include: review of plans, profiles and cross-sections; review of available geotechnical data; and field reconnaissance. As also previously discussed, the proposed laboratory testing program must be documented in the “Subsurface Exploration Planning Submission”. Requirements for this report are included in [Chapter 1](#) of this publication. This report must be reviewed and approved by the DGE prior to performing the laboratory testing, and any modifications to the originally approved laboratory testing program because of findings from the subsurface investigation must be approved by the DGE prior to testing.

Exact laboratory testing requirements cannot be specified in this publication due to the multiple variables associated with a project, including: size of project, complexity of project, design elements (e.g., roadway, pavement, structure), type(s) of strata, thickness of strata, consistency of strata, and others. Some basic requirements are provided herein, along with guidelines and considerations that should be helpful for developing a laboratory testing program tailored to the specific project.

Laboratory testing is required when borings are performed for a project during the Preliminary and Final Design Phases unless otherwise approved by the DGE. At a minimum, moisture content and classification tests (i.e., gradation and liquid/plastic indices) must be performed to verify field descriptions. The number of tests performed and the need to perform

other type(s) of tests will be dependent upon the size of the project, the proposed construction, and the complexity of the subsurface conditions.

If borings are performed during the Alternatives Analysis Phase of a project, a very limited laboratory testing program will most likely be sufficient. The Alternatives Analysis Phase laboratory test program should include a limited number of laboratory classification tests. Other testing (e.g., corrosion, rock strength, acid-base accounting, etc.) should be performed if it will be helpful to evaluate the proposed alignment alternatives. Laboratory testing during the Preliminary and Final Design Phases will be more comprehensive to help perform geotechnical analyses and provide recommendations.

The laboratory testing program must be developed at the same time the subsurface exploration program is developed. Similar to the subsurface exploration program, the laboratory testing program is developed using available information, which at a minimum should include: plans, profiles and sections of proposed construction; published soils and geologic information; and field reconnaissance observations. The proposed laboratory testing program must be presented in the “Subsurface Exploration Planning Submission”. Indicate the type(s) and numbers of test proposed, and discuss the purpose and benefit of each type of test.

Keep in mind that, in many cases, the proposed laboratory testing program presented in the “Subsurface Exploration Planning Submission” will be developed with no to minimal site specific subsurface information. Therefore, once the test borings are drilled, the proposed laboratory testing program may have to be modified if the actual subsurface conditions vary from those that were anticipated. Any changes to the originally proposed laboratory testing program must be discussed and approved by the DGE.

Another reason the laboratory testing program must be developed with the subsurface exploration program is to ensure that the appropriate number and type(s) of samples are collected during the subsurface exploration. For example, if CBR and/or moisture density relation testing is proposed, collection of bulk samples must be included in the subsurface exploration program. If consolidation or shear strength testing is proposed, collection of undisturbed samples (i.e., Shelby tubes) must be included in the subsurface exploration program.

4.3 GUIDELINES FOR SELECTING LABORATORY TESTS AND SOIL AND ROCK SAMPLES

As discussed above, the laboratory testing program is initially developed during the planning of the subsurface exploration program, and then modified as needed based on the actual subsurface conditions encountered by the subsurface exploration (i.e., test borings and test pits). Below are some guidelines and considerations that can be used to develop the initial laboratory testing program, to modify the program as needed once the site specific subsurface conditions are determined, and to select the appropriate soil and rock samples for laboratory testing.

1. Plot the boring information on the roadway profile, structure elevation view, etc. to help understand soil/rock stratification. In addition to field descriptions, look for similarities in color and density/consistency to help group possible stratum

that have slightly different field descriptions. Include on the subsurface profile the proposed roadway grade and/or bottom of structure footing elevations to approximate their relationship to the soil/rock strata. Preferably these profiles should be developed by the drilling inspector or office personnel as borings are completed so samples for laboratory testing can be selected by the PGM and retrieved as quickly as possible.

2. Consider the proposed construction and the anticipated geotechnical analyses and recommendations that will be needed. Perform testing needed to appropriately select the soil and rock parameters needed for the geotechnical analyses that are anticipated to be performed.
3. Test soil samples from the various strata to adequately verify field descriptions and identify necessary soil index and engineering properties. When approximately four or fewer split-barrel samples are collected in a stratum, combine all the samples to perform laboratory testing. The combining of soil samples will often make it easier to update the Final Engineer's Log with the results compared to just testing a few of the samples. Additionally, since there will be some variation among individual samples obtained from a stratum, it is best practice to combine as many samples as possible/practical for laboratory testing to attain representative results for the stratum.
4. For structure foundations, concentrate laboratory testing on soil stratum/samples that are significant to the geotechnical design. For proposed spread footings bearing on soil, concentrate the soil testing within the zone of influence of footing stresses, which is approximately two times the footing width beneath the footing. Shear strength testing should almost always be performed when spread footings on soil are proposed. For spread footings bearing on bedrock and end/point bearing piles driven to bedrock, soil testing will typically not be critical, although classification testing to verify field descriptions is still necessary, and corrosion testing will be needed in most cases when deep foundations are proposed. For proposed friction piles, laboratory testing including, at a minimum, classification and shear strength, is critical. If temporary shoring is anticipated, ensure sufficient soil classification and/or rock compressive strength tests are performed for selection of temporary shoring parameters. In some cases, soil shear strength testing may also be appropriate for selection of temporary shoring parameters.
5. For roadway design, concentrate laboratory testing on soil stratum/samples that are significant to the geotechnical design. Test pavement subgrade soils, soils within the zone of influence of embankment stresses, soils in proposed cut slopes, and soils that will be excavated and reused to construct embankments. Minimal testing to no testing is typically required on excavated material that will be wasted off site. An exception to this would be if wasted material needs to be tested for acid bearing potential.

6. Minimal testing to no testing should typically be performed on samples collected within a few feet of ground surface or at depths beyond the influence of embankment or footing stresses. Structure footings are typically placed several feet below ground surface for frost protection, and soils within a few feet of ground surface are usually stripped/removed or compacted prior to embankment placement; therefore, these soils are typically not critical to the geotechnical design.
7. More testing is typically required on fine-grained soils compared to coarse-grained soil. Coarse-grained samples are generally more accurately described in the field, and their behavior is more predictable than fine-grained soil. Fine-grained soils are often less favorable for construction compared to coarse grain soils; therefore, it is important to have a good understanding of the gradation and plasticity of fine-grained strata. Additionally, shear strength and consolidation testing of fine-grained soils are often needed to estimate these engineering properties since presumptive values are very unreliable.
8. Projects with horizontal/uniform subsurface profiles will typically require less laboratory testing compared to non-uniform profiles.
9. Shear strength testing should typically be performed when spread footings bearing on soil or friction piles are proposed, and when slope stability is a concern.
10. Consolidation testing should be performed when embankments are proposed to be placed on fine-grained, compressible soil, and the compressible stratum is within the stress influence of the embankment. It should also be performed when pile downdrag is a concern. Typically, soil must be saturated (i.e., below groundwater) and very soft to stiff consistency (i.e., N-value less than or equal to 15 blows per foot) to be prone to significant consolidation settlement.
11. The number of tests, particularly shear strength and consolidation, performed for a project will be dependent upon project specific conditions. Performing just one of these tests can be problematic because the validity of the results cannot be compared to other test results. If two tests are performed, and the results vary significantly, it may be difficult to determine which result is representative. Three or more tests are useful for comparison of test results and selection of design parameters; however, this number of tests may not be appropriate for all projects. For example, on small, less complex projects, performing just one test may be appropriate. Consider a single span bridge replacement project where the subsurface conditions are uniform, and shear strength testing is performed on remolded granular soil for spread footing design. One shear strength test may be appropriate because the test result can be compared to internal angle of friction correlations with SPT to determine the reasonableness of the test results. Conversely, for a new, multi-span bridge where spread footings and friction piles are being considered, three or more shear strength tests would more likely be appropriate.

12. California Bearing Ratio (CBR) and moisture-density testing should be performed when pavement design is required, and the pavement subgrade material is known (i.e., in-situ subgrade or embankments constructed with on-site excavated soils). CBR testing is not appropriate for projects where the pavement subgrade material is not known, such as a project with embankments constructed from an unknown borrow source. CBR testing is also not necessary for projects where the proposed pavement section is selected based on matching the existing pavement section.
13. Unconfined compressive strength testing of rock should be performed for foundations bearing on bedrock, including: spread footings, drilled shafts and micropiles. Unconfined compressive strength testing of rock should also be performed for linear type retaining walls socketed into bedrock and when rock anchors are proposed. This testing should also be considered for proposed rock cuts to help evaluate rock excavation.
14. Slake Durability testing should be considered for shallow and deep foundations founded on, and rock cuts or excavations within, claystone, shale and similar weak rocks to estimate qualitatively the durability of weak, non-durable rocks.
15. Acid-Base Accounting testing should be performed on rock and/or soil that are located within formations known to contain acid producing soil/rock, and on samples identified in the field that show signs of acid producing minerals.

4.4 DISTURBED, UNDISTURBED, AND REMOLDED SOIL SAMPLES

Laboratory testing can be performed using disturbed and undisturbed soil samples collected from the subsurface exploration. Additionally, for tests that require the soil sample to be a specific size/shape for the testing apparatus, samples remolded from disturbed samples can be used, if appropriate. Determining which type of sample to use for various tests will depend on numerous factors, including: test to be performed, soil type, field/construction condition being modeled by the test, and types of samples collected during the subsurface exploration.

4.4.1 Disturbed Soil Samples

Disturbed soil samples, which include samples collected from a split-barrel sampler, augers, and test pits, are best suited for laboratory index tests. Index tests include natural water content, gradation, liquid and plastic limits, and others, which are listed in [Section 4.8.4](#). Disturbed soil samples are more economical to collect compared to undisturbed samples; therefore, they should generally be used for tests that do not require an undisturbed sample. Additionally, SPT's performed while collecting split-barrel samples provide an indication of density/consistency, and this complementary information is not obtained while collecting undisturbed samples. Disturbed soil samples cannot be used for consolidation testing, and should only be used for shear strength testing under the proper circumstances (see discussion below on remolded samples).

4.4.2 Undisturbed Soil Samples

Undisturbed soil samples are required when conducting performance tests (i.e., consolidation tests, and some shear strength and permeability tests). Undisturbed samples should be used anytime the goal of the testing is to simulate the in-situ or natural, in-place soil conditions. Undisturbed samples can be used to test soil index properties; however, unless these index tests are performed in conjunction with performance tests, disturbed samples should generally be used because it is more economical.

Undisturbed samples are typically collected with a Shelby tube sampler. Shelby tube samplers can generally be used in fine-grained soils that are very soft to stiff (i.e., SPT N-values less than 15 blows per foot). If soil conditions do not permit sample collection with a Shelby tube sampler (e.g., stiff/dense, high gravel content, etc.), relatively undisturbed samples can be obtained with either a Pitcher or Denison tube sampler. A detailed discussion of these various tube samplers is included in [Chapter 3](#) of this publication.

When undisturbed samples are collected, it is best practice to obtain samples in addition to what is believed to be needed for testing. The quality of undisturbed samples cannot be determined until the sample is extruded in the laboratory, so additional samples should be available in case sample quality is found to be poor upon extrusion. Also, if test results appear to be erroneous or suspect, additional samples should be on hand so more testing can be performed.

4.4.3 Remolded Soil Samples

Some geotechnical tests, including shear strength and permeability, require the soil sample be a specific size/shape for the testing apparatus (i.e., direct shear box, triaxial or permeameter cell). Undisturbed soil samples from tube samplers are typically ideal for these tests when modeling existing conditions. However, when it is not possible to collect undisturbed samples, samples remolded from disturbed soil samples can be used. When modeling conditions where soils are intended to be disturbed, remolded and compacted, only remolded samples (not undisturbed samples) should be used for testing.

The remolding process cannot necessarily simulate the in-situ (undisturbed) soil condition (i.e., density/consistency and soil structure), particularly for fine-grained soils. In such cases, test results must be evaluated thoroughly by the design engineer to assure the results obtained are consistent with the materials and site/project conditions. When appropriate, values can be compared to test results from published literature (of the same materials that were prepared and tested in a manner modeling the same conditions) to aid in assessing the validity of the results prior to use in analyses or design. For example, residual soils may contain relic rock structure that increases the shear strength of the in-situ soil, and this relic rock structure cannot be duplicated with a remolded sample. Alternatively, glacial deposits may contain varves, or annual sediment layers that reduce the shear strength of the in-situ soil, and cannot be duplicated with a remolded sample.

Laboratory test samples remolded from disturbed samples are appropriate for shear strength testing of granular soil. Granular soil can generally be remolded to closely simulate in-

situ conditions. Additionally, it is typically not possible to obtain undisturbed samples of granular soil due to lack of cohesion. Samples remolded from disturbed samples are also appropriate for shear strength testing of soil that will be placed and compacted during construction (e.g., embankment fill, select fill, etc.). Since relic structure, varves, etc. will not exist in the compacted fill, the remolded laboratory samples will most likely be representative of the field conditions.

4.5 LABORATORY TESTING PRECAUTIONS

Precautions must be taken when performing laboratory testing to help ensure that the test results are representative of the in-situ conditions. These precautions must be taken in the field while collecting, packaging, and transporting the samples, and in the laboratory while storing, handling and testing the samples.

4.5.1 Collecting Samples

The correct tools (e.g., split-barrel sampler, Shelby tube, core barrel, etc.) and sampling procedures, as indicated throughout Publication 222, Subchapter 5E, must be used when collecting both disturbed and undisturbed soil and rock samples. The drilling inspector must verify the depths at which the samples are being collected. Sampling depth can easily be determined by measuring the length of drill tools/rods placed in the boring and subtracting the length of drill rod “stick up” above ground surface. Additionally, it is critical that the boring is cleaned (e.g., free of disturbed material) prior to sampling to ensure that the sample collected is representative of the material at the reported depth.

4.5.2 Sample Volume and Number of Samples

Ensure that an adequate volume or numbers of samples are collected during the subsurface exploration, particularly undisturbed and bulk soil samples, such that both the needs for core box record and all potential laboratory testing are satisfied. Relative to undisturbed samples, multiple sample increments should be collected beyond those that are anticipated to be needed for testing. When extruded in the laboratory, undisturbed samples may be found to be damaged or inadequate for testing, so obtaining additional samples may be needed for testing. Also, sometimes laboratory test results appear erroneous or may not be as expected. In these cases, additional testing, and therefore additional samples, will be needed. The cost of obtaining additional samples during the subsurface exploration is significantly less than the cost of remobilizing drilling equipment to obtain additional samples, or the potential costs and problems associated with performing design without adequate information.

4.5.3 Packaging Samples

Packaging soil and rock samples properly is very important in ensuring representative laboratory test results. Procedures indicated in Publication 222, Section 215 must be followed. All samples must be clearly and accurately labeled to ensure the proper samples are tested. Disturbed soil samples must be kept in sealed jars to retain the natural moisture content. Ensure that the jar top and lid are clean to provide a good seal, and do not overfill jars. When bulk soil

samples are collected, place a representative sample in a sealed jar within the bag for moisture content testing. Shelby tubes and other undisturbed soil samples must be properly capped, sealed with wax and taped to help secure the sample within the tube and retain the natural moisture content. Undisturbed soil samples must also be carefully transported, as indicated in [Section 4.5.4](#).

Depending upon the type of rock, core samples may have to be “specially” packaged. Fine-grained rocks, like claystone, mudstone, shale, siltstone, etc. should be wrapped in moist paper towels and plastic wrap to retain the natural moisture and prevent deterioration prior to laboratory testing. Rock samples should be transported with care to prevent breakage, and bubble wrap or other cushioning material should be considered for protection. Once the rock core is removed from the boring, the natural confining stress is removed, and the core may break along weakly cemented joints. Wrapping the core in moist towels, plastic wrap, bubble wrap, tape, etc. can help to prevent this breakage prior to laboratory testing.

4.5.4 Storing Samples

Storing samples properly is also important to obtain quality laboratory test results. Soil and rock core samples should be stored in a location to prevent freezing or exposure to extreme heat. If jar samples freeze, the jars may break which will affect the natural moisture content and may “contaminate” the soil and prevent testing. Similarly, if rock core samples freeze, moisture within the sample may cause the core to break. Undisturbed soil samples are extremely sensitive and must also be transported and stored in an upright position (i.e., same orientation as in the ground), ideally in a Shelby tube box, and protected from vibration, dropping, rolling, etc. The drilling inspector should take possession of undisturbed samples immediately after the driller has preserved the sample, and deliver to the laboratory as soon as possible. Undisturbed samples must not be stored in a vehicle. Alternatively, temporarily store these samples in a secure location on site or off site (i.e., office, home, hotel/motel room, etc.).

There is no exact time limit for how long samples can be successfully stored prior laboratory testing. The acceptable length of storage time will be dependent upon sample type, the testing to be performed, and how the sample is preserved/stored. Samples should be tested as soon as practically possible, and in particular, undisturbed samples, samples that are tested for natural moisture content, and “weak” rock samples. Soil and rock samples to be laboratory tested should be selected while the borings are being performed or immediately after they are drilled, and promptly delivered to the laboratory for testing.

The Publication FHWA-NHI-01-031, “Subsurface Investigations – Geotechnical Site Characterization” does not recommend long term storage of soil samples in Shelby tubes. Soil can cause the tubes to corrode, which leads to adhesion of the soil to the tube. This adhesion can make it difficult to extrude the sample, and microscopic failures in the soil can develop during extrusion. If undisturbed testing is performed on these samples, the results may be erroneous. This FHWA publication also indicates that research has shown that undisturbed samples stored more than 15 days can experience significant changes in strength characteristics and result in unreliable test results.

4.5.5 Visually Review Samples before Testing

Avoid preparing laboratory test work orders/requests based solely on the boring logs. The PGM should visually inspect the samples selected for laboratory testing prior to laboratory testing to ensure that field descriptions appear accurate and revisions to the laboratory test work order/request are not needed. This should prevent unnecessary testing, and ensure that the testing needed for the geotechnical design is performed in a timely manner. If laboratory testing is being done off-site, and the PGM cannot visually inspect the samples, the PGM should discuss the testing with the laboratory testing manager to verify that the laboratory work order/request appears reasonable. Additionally, when testing is performed on a soil sample from a Shelby tube, the PGM should inspect the sample after extrusion to direct the laboratory on which part(s) of the sample to use for testing. If testing is being performed off-site, digital photographs of the extruded sample can be used by the PGM to direct the laboratory testing.

4.5.6 Selecting Samples from Shelby Tubes

As previously discussed in [Chapter 3](#), Shelby tubes are the most common method in Pennsylvania to obtain undisturbed soil samples to preserve the in-situ nature of the soil to the best extent possible and to estimate in-situ soil properties. When testing samples extruded from Shelby tubes, be sure to select representative portions of the sample for testing. Sometimes the material recovered in the tube will not be consistent throughout the full length of the recovered sample, and often the tendency is to select the “worst” (e.g., softest, weakest, etc.) portion of the tube for testing. This approach is reasonable if the portion of the sample selected is representative of the majority of the tube. It is not reasonable/appropriate to test a portion of the sample that is unrepresentative of the majority of the tube and apply the test results to an entire soil stratum. Doing this can result in overly conservative and costly design recommendations. If it is believed that the localized “worst” portion of the tube is critical to the design (for instance, if it represents the failure plane of a landslide), consult with the DGE to determine if additional testing is necessary.

4.6 VERIFICATION OF LABORATORY TEST RESULTS

It is critical that the PGM review the laboratory test results immediately as they become available to ensure that they are consistent with expectations. If the results are not consistent with expectations, if they vary significantly between what are believed to be similar materials, or if the results appear not to be representative due to sample disturbance, laboratory error, etc., the PGM must determine if additional testing is required. This determination must be made quickly so that additional samples can be selected for testing, and the testing can be performed without adversely impacting the design schedule. If additional laboratory budget is needed to perform the testing, the PGM must have approval from the DGE prior to performing the testing. Ideally, sufficient samples, and in particular, undisturbed and bulk samples, should be collected during the subsurface exploration to permit “unforeseen”, additional testing. If additional drilling is needed to obtain the necessary samples for testing, the DGE must also approve of this work before it is performed.

4.6.1 Example Projects

As previously indicated, exact testing requirements cannot be specified in this publication due to the multiple variables of a project. Keeping this in mind, below are examples of typical Department roadway and bridge construction projects, and what a laboratory test program may consist of based on the conditions indicated.

Example 1

Box culvert replacement with no roadway widening or vertical profile change: box culvert is 75 feet long, new box is approximately same size as existing, wing walls are 15 feet long and new pavement will match existing. Assume three borings were drilled (one at inlet, one near mid length and one at outlet) and one pavement core.

Subsurface conditions: Subsurface conditions will generally have little impact on this design/construction since it is a replacement with no profile increase or widening (i.e., no to minimal increase in load on the foundation). A relatively simple laboratory testing program is appropriate and may include:

- Four each – natural water content, sieve/gradation and liquid and plastic limits (i.e., classification)
- One soil corrosion series (pH, resistivity, sulfate and chloride)
- One water corrosion series (pH, conductivity/resistivity, sulfate and chloride)

Note: Shear strength testing typically is not needed for box culverts since bearing resistance rarely controls the design. Consolidation testing of saturated, cohesive soil may be needed if the culvert increases the load on the foundation, such as, if the culvert is constructed for a new roadway alignment or a roadway widening, or if the profile grade of an existing roadway is being raised considerably.

Example 2

Two-span bridge replacement with minor approach embankment widening: bridge spans a roadway (pier in the median); existing foundations are unknown; 50 feet of minor approach embankment widening on each side of the bridge; 15-foot high approach embankments; 5-foot (max.) widening on each side of embankment; and new pavement to match existing pavement section. Assume eight borings were drilled (two borings for each substructure and one boring on each side of bridge at toe of existing fill in proposed embankment widening area) and two pavement cores were obtained.

Subsurface conditions: Assume borings indicate relatively uniform conditions across the site, including: shallow bedrock (less than 10 feet below median grade), loose to medium dense granular overburden soil, and medium to very stiff sandy and gravelly clay embankments. Proposed foundations include abutments (stub or integral) on H-piles and pier spread footing bearing on bedrock.

Since the subsurface conditions are generally uniform and consist of granular soil, and the foundations are not complex, a relatively simple laboratory testing program is appropriate and may include:

- Eight each - natural water content, sieve/gradation and liquid and plastic limits (i.e., classification)
- Two soil corrosion series (pH, resistivity, sulfate and chloride) from abutment borings
- Five bedrock unconfined compressive strength tests from pier borings, or 20-point load breaks if unconfined compressive strength tests cannot be performed

Testing justification: Water content tests should generally be performed to compliment other testing. The eight classification tests should be adequate to verify field descriptions. Soil corrosion test results will be used for abutment pile design, and bedrock strength testing will be used for pier foundation design. Since soils are granular and foundations do not attain axial resistance from the soil, soil strength testing is not needed. Additionally, embankment height and widening is minimal so stability analysis is not needed.

Example 3

Same bridge project as *Example 2* above except subsurface conditions consist of thick (50 feet or more), predominantly granular overburden overlying bedrock. Proposed foundations alternatives include abutments (stub or integral) on friction or end/point bearing piles, and pier on spread footing on soil or piles (friction or end/point bearing).

Compared to *Example 2* above, the subsurface is slightly more complex since bedrock is deep, and the foundations are more complex since friction piles and a spread footing on soil are being considered. Consequently, more laboratory testing will be required and it may include:

- Twelve each - natural water content, sieve/gradation and liquid and plastic limits (i.e., classification)
- Two soil corrosion series (pH, resistivity, sulfate and chloride)
- Three shear strength tests (direct shear or triaxial shear)

Testing justification: Water content tests should generally be performed to compliment other testing. The twelve classification tests should be adequate to verify field descriptions. Soil corrosion test results will be used for pile design. Shear strength testing of soil is needed since friction piles and a spread footing on soil are being considered.

Notes:

1. If soft, saturated cohesive soil is present, consolidation testing is most likely not needed since minimal increased load will be placed on the soil since this is a bridge replacement project with minimal embankment widening. The presence of this soil in the subsurface will most likely require point/end bearing piles driven to bedrock in lieu of friction piles or spread footings. Consequently, shear strength testing will not be needed.

2. If bridge spans a waterway (e.g., stream, creek, river, etc.), water corrosion testing (pH, conductivity/resistivity, sulfate and chloride) is needed.

Example 4

Two-span bridge replacement/realignment: bridge replaces an existing structure, but on a new alignment; bridge spans a roadway (pier in the median); existing foundations are unknown; 200 feet of approach embankment realignment on each side of the bridge; 15-foot high approach embankments; and new pavement to match existing pavement section. Assume eight borings were drilled (two borings for each substructure and one boring on each side of bridge beneath the proposed approach embankment) and two pavement cores were obtained.

Testing for this scenario would be similar to that discussed in *Example 3* above. However, due to the realigned embankments, consolidation testing would most likely be required if soft, saturated cohesive soil is present since these areas were not previously loaded by embankment.

4.7 PRESENTATION OF LABORATORY TEST RESULTS

It is important that laboratory test results are presented and documented adequately and consistently in boring logs, reports, plans, profiles, cross-sections and contract documents. Proper presentation and documentation provides a permanent record of the laboratory testing, allows the designer and reviewer(s) to better understand the laboratory test results and how they influence geotechnical analyses and recommendations, and gives contractors additional soil/rock information for consideration during bidding/planning of work.

The following are requirements for the presentation/documentation of laboratory test results in various documents. The requirements must be followed unless directed otherwise by the District Geotechnical Engineer (DGE).

4.7.1 Final Engineer's Log

Laboratory AASHTO and USCS classification symbols must be shown on the Final Engineer's Log. As indicated in Publication 222, Chapter 3.6.3, field estimated USCS and AASHTO symbols are denoted on the boring log with lower case letters. When laboratory classification testing is performed, the Final Engineer's Log must indicate which strata were tested by using upper case letters for these symbols.

4.7.1.1 Overview of Final Engineer's Log

One of the main goals of the Final Engineer's Log is to identify the strata (i.e., soil and rock layers) beneath a project site, and it is important to keep in mind that within each stratum of soil and rock there will typically be some degree of variation. For example, a soil stratum may be comprised primarily of lean clay with sand and gravel. The amount/percentage of clay, sand and gravel in each split-barrel sample obtained from this stratum will vary, but this does not mean each individual sample is its own unique stratum. Similarly, the color of the individual

split-barrel samples may have some variation, and the plasticity of the samples will vary somewhat, but again this does not mean each sample is its own unique stratum. Therefore, although detailed information should be presented on the Final Engineer's Log, do not lose sight of the overall goal and purpose of the log. A Final Engineer's Log that denotes numerous, thin soil strata that have only slight variations is not the intent of the log, is typically not useful, and often creates unnecessary work due to the complexity of the log.

Unlike previous versions of the Final Engineer's Log, the current log does not include an entry for a field AASHTO and USCS symbol for each split-barrel soil sample that is collected in a boring. Instead, these symbols are only shown for each soil stratum identified/described, and each soil stratum will typically include two or more split-barrel samples. Additionally, the method to describe soil in Publication 222, Chapter 3.6.3 for field descriptions is adapted from the Burmister Soil Description System and it is not related to the AASHTO or USCS classification system. Therefore, multiple AASHTO and USCS symbols can apply to the same field description. For example, a soil stratum field described as CLAY and SILT, would most likely include AASHTO symbols A-6 and A-7-6, but could also include A-4, A-5 and A-7-5. Similarly, the USCS symbol associated with this field description would most likely be CL, but it could also include ML. Therefore, when preparing the Inspector's Field Log, keep in mind that it is expected/acceptable to have some variation in the soil within each stratum, and this can include variation in the AASHTO and/or USCS symbols. Furthermore, when field and laboratory AASHTO and USCS classification symbols do not agree, it is not acceptable to simply add a stratum to the log for the samples that were tested and keep the stratum that was described in the field for the samples that were not tested. It is the PGM's responsibility to review all the information (i.e., field and laboratory) and decide how to appropriately incorporate the laboratory information on the Final Engineer's Log. Several examples of updating a boring log with laboratory test results are provided below.

4.7.1.2 Incorporating Laboratory Classifications onto the Final Engineer's Log

As discussed above, laboratory AASHTO and USCS symbols must be shown on the Final Engineer's Log in capital letters. A laboratory classification is more precise than a field description, and in order to clearly present the subsurface conditions, the field description on the Final Engineer's Log must at least closely resemble the laboratory classification. If they do not closely resemble one another, the PGM must investigate the discrepancy and make revisions as appropriate. There are several scenarios that may occur with respect to the Final Engineer's Log and laboratory classification test results. Some of these examples and guidelines for handling them are discussed below, and a brief discussion is also included in Publication 222, Chapter 3.6.2 under the fifth bullet.

Example 1

Assume four consecutive split-barrel samples are collected from a test boring, the samples are similar, and this stratum is field described as F SAND and CLAY, some silt (a-6/sc). This field description would indicate that there could be approximately equal amounts of fine SAND and CLAY since the descriptor "and" (i.e., content $\geq 35\%$ per Publication 222, Chapter 3.6.3(a)) was used. These are the only four samples collected from this stratum, the samples are combined to

perform one laboratory classification test, and the test indicates that the combined samples classify as they were described in the field (approximately equal amounts of sand and clay). Since the field and laboratory symbols are the same, change the lower case AASHTO and USCS letters to capital letters, and where applicable add the group number, on the Final Engineer's Log (i.e., A-6(8)/SC). Note that gINT will automatically update the AASHTO and USCS symbols on the Final Engineer's Log once the laboratory test results are entered into gINT. Also, note that the field description may need to be changed depending upon the relative amounts of the various constituents.

Example 2

Assume the same situation as above, except the USCS laboratory classification is CL instead of SC. Similar to above the AASHTO and USCS symbols from the laboratory test results (i.e., A-6(8)/CL) must be shown on the Final Engineer's Log in capital letters. Since the laboratory gradation analysis indicates there is considerably more clay than sand, the field description must be changed to be consistent with the laboratory classification (i.e., CLAY, some fine Sand, some Silt).

Example 3

Assume eight consecutive split-barrel samples are collected from a test boring, the samples are similar, and this stratum is field described as CLAY, little SAND (a-7-6/ch). The middle four of the eight samples are combined to perform one laboratory classification test, and the test indicates that the combined samples classify as A-6/CL. The laboratory test results must be reviewed to determine how to appropriately modify the Final Engineer's log. A change in the field/visual description on the Final Engineer's Log may be required. If the field description is inconsistent with the laboratory classification, then the description on the Final Engineer's Log must be modified to be consistent with the laboratory classification. Furthermore, if the field/visual description for adjacent materials that were not laboratory classified is the same as the laboratory classified material, then the field/visual description of the non-laboratory classified material must be changed to a field/visual description (i.e., lower case letters) consistent with the laboratory classified material. No change is required if the field/visual description for adjacent differs from the field/visual description of the laboratory classified materials such that the original classifications of the adjacent materials are more representative of the field/visual description.

4.7.2 Geotechnical Engineering Reports

Laboratory test results must be thoroughly documented in all geotechnical engineering submissions/reports as indicated in [Chapter 1](#).

4.7.3 Soil Profile for Roadway Plans

Laboratory test results must be graphically presented on the roadway profile according to DM-3, Chapter 5.

4.7.4 Drafted Structure Borings

As indicated in DM-4 Section 1.9.3, the information on the Final Engineer’s Log must be presented on the drafted structure borings accompanying the bridge/structure plans. Publication 222 requires the use of Bentley gINT® Software and the Department’s gINT Library to produce the Final Engineer’s Logs. Use gINT Software in conjunction with the structure boring and laboratory test summary report formats available in the Department’s gINT Library to prepare structure borings. Include soil and rock core laboratory test result tables on the structure borings.

4.8 GEOTECHNICAL SOIL, ROCK, AND WATER LABORATORY TESTS

The following sections provide information and guidance for various geotechnical soil, rock, and water laboratory tests.

4.8.1 AASHTO Materials Reference Laboratory (AMRL)

Laboratory testing must be performed by an AASHTO Materials Reference Laboratory (AMRL) accredited laboratory. It should be noted that AMRL certifies/assesses laboratories for specific tests, not the laboratory in its entirety. Therefore, when selecting a laboratory for testing services, ensure that the laboratory is accredited for each individual test that is proposed. Note that most geotechnical related laboratory tests are accredited by AMRL, but not all tests. In cases where AMRL does not certify a specific test proposed, any reputable laboratory may be used to perform these tests. A summary of geotechnical laboratory tests is included in [Section 4.8.4](#), and the summary indicates whether each test is accredited by AMRL. A listing of laboratories with their respective accredited test methods, and other information about AMRL can be found on the [AMRL website](#). The AMRL program has recently been renamed. It is now referred to as AASHTO resource.

AASHTO assessments are performed in 2 ways. For selected tests, i.e. classification, compaction and California Bearing Ratio (CBR) tests, laboratories conduct tests on prepared samples provided by AASHTO and the results are evaluated. This is referred to as the Proficiency Sample Program (PSP). For more complex tests, such as consolidation and strength, an auditor observes the lab technician for all or portions of the test, often concurrent with an unofficial oral examination. The PSP program provides testing data from about 3,000 testing laboratories on samples prepared by AASHTO to be of uniform composition. Conclusions can be made based on the overall results of the PSP process. The following table presents the Coefficient of variation (CV) of PSP data sets for tests included in the PSP process. Coefficient of variation is the ratio of the standard deviation to the mean. In other words, it is the variation of a data set with respect to its mean. The purpose of [Table 4.8.1-1](#) is to provide an indication of the variability and repeatability of various soil laboratory tests. For example, the maximum dry

density and optimum moisture content for moisture density testing show very low standard deviations and coefficients of variation. When looking at Sample 169 for max dry density, we see a standard deviation of 1.5 for a maximum dry density average of 124.5 lb/ft². In this case, the standard deviation is only 1.2% of the maximum dry density value. However, when looking at CBR, it is observed that the standard deviations and coefficient of variations are very high. When looking at Sample 169 for CBR, we see a standard deviation of 5.4 for a CBR average of 11.1. In other words, the standard deviation is 49% of the average CBR value. Clearly CBR testing does not provide the consistency or repeatability of moisture density testing. This is the way that [Table 4.8.1-1](#) can be used to assess the repeatability of lab test data for parameter selection. As shown below, CBR test results and hydrometer test results for clay sized particles exhibit the highest variability in results.

Table 4.8.1-1 – AASHTO Proficiency Sample Program (PSP) for Laboratory Soil Tests¹ with Data Variability

Test	AASHTO Designation	Equivalent ASTM Designation	Classification AASHTO Sample No.	Total No. Labs	Units	Average	Standard Deviation, S	Coefficient of Variation, CV (%)
Particle Size Analysis of Soils by Hydrometer								
Total material passing the No. 40 Sieve	T88	D422	169	1209	percent	65.28	3.09	4.7
			170			68.71	3.04	4.4
			171	1216		74.73	2.03	2.7
			172			73.98	1.97	2.7
			173	1256		72.09	2.27	3.1
			174			67.50	2.53	3.7
Total material passing the No. 200 Sieve			169	1215		53.08	2.94	5.5
			170			57.58	2.96	5.1
			171	1220		52.55	1.49	2.8
			172			51.94	1.35	2.6
			173	1262		48.17	2.13	4.4
			174			44.53	2.28	5.1
Total material smaller than 0.02 mm	169	1118	46.38	4.29	9.2			
	170		50.08	4.29	8.6			
	171	1132	47.42	3.49	7.4			
	172		46.69	3.47	7.4			
	173	1176	39.10	3.74	9.6			
	174		36.50	3.69	10.1			

Test	AASHTO Designation	Equivalent ASTM Designation	Classification AASHTO Sample No.	Total No. Labs	Units	Average	Standard Deviation, S	Coefficient of Variation, CV (%)			
Particle Size Analysis of Soils by Hydrometer (cont.)											
Total material smaller than 0.002 mm	T88	D422	169	1115	percent	21.73	3.34	15.4			
			170			26.30	3.34	12.7			
			171	1129		22.91	3.42	14.9			
			172			22.88	3.36	14.7			
			173	1172		13.71	2.71	19.8			
			174			12.79	2.63	20.6			
Total material smaller than 0.001 mm			169	1104		15.28	3.19	20.9			
			170			20.04	3.18	15.9			
			171	1122		15.56	3.15	20.2			
			172			15.83	3.18	20.1			
			173	1163		8.79	2.83	32.2			
			174			8.29	2.78	33.5			
Atterberg Limits											
Liquid Limit of Soils	T89	D4318	169	1494	percent	32.46	2.03	6.3			
			170			54.73	3.51	6.4			
			171	1525		26.77	2.02	7.5			
			172			29.53	2.14	7.2			
			173	1587		26.11	1.36	5.2			
			174			25.99	1.35	5.2			
Plastic Limit of Soils			T90	D4318		169	1489	percent	17.46	1.34	7.7
						170			17.58	1.83	10.4
						171	1524		14.13	1.17	8.3
						172			14.25	1.23	8.6
						173	1587		14.86	1.02	6.9
						174			14.77	1.06	7.2
Specific Gravity of Soils											
Passing 2.0 mm	T100	D854			169	897	N/A		2.66	0.029	1.1
					170				2.66	0.034	1.3
					171	876			2.66	0.028	1.1
					172				2.66	0.030	1.1
					173	919			2.66	0.027	1.0
			174	2.66	0.025			0.9			

Test	AASHTO Designation	Equivalent ASTM Designation	Classification AASHTO Sample No.	Total No. Labs	Units	Average	Standard Deviation, S	Coefficient of Variation, CV (%)
Moisture Density (Proctor) of Soils								
Optimum Moisture Content - Standard	T99	D698	169	1211	percent	9.7	0.83	8.5
			170			11.7	1.08	9.2
			171	1208		9.3	0.59	6.4
			172			9.7	0.56	5.8
			173	1283		8.2	0.58	7.1
			174			8.0	0.56	7.0
Maximum Dry Density - Standard	T99	D698	169	1211	pcf	124.5	1.52	1.2
			170			118.1	1.86	1.6
			171	1208		126.7	1.31	1.0
			172			125.5	1.22	1.0
			173			129.2	1.41	1.1
			174			129.8	1.44	1.1
Moisture Content of Soils								
Moisture Content Immediately Before Compaction	T265	D2216	169	516	percent	9.9	0.19	1.9
			170			9.0	0.18	2.0
			171	521		9.4	0.15	1.6
			172			9.0	0.14	1.6
			173	535		9.0	0.13	1.5
			174			8.5	0.13	1.5
Moisture Content of Unused Material Immediately After Compaction	T265	D2216	169	516	percent	9.8	0.23	2.3
			170			8.9	0.20	2.3
			171	520		9.4	0.16	1.7
			172			8.9	0.17	1.9
			173	536		8.9	0.16	1.8
			174			8.4	0.15	1.8
CBR								
Dry Unit Weight of compacted specimen before soaking	T193	D1883	169	516	pcf	126.4	1.40	1.1
			170			129.7	1.23	0.9
			171	521		120.3	1.12	0.9
			172			125.7	1.03	0.8
			173	536		123.3	1.12	0.9
			174			127.9	1.25	1.0

Test	AASHTO Designation	Equivalent ASTM Designation	Classification AASHTO Sample No.	Total No. Labs	Units	Average	Standard Deviation, S	Coefficient of Variation, CV (%)	
CBR (cont.)									
Swell - Percentage of Initial Specimen Height	T193	D1883	169	512	percent	-0.160	0.20	-122.5	
			170			-0.115	0.16	-142.6	
			171	521		-0.011	0.04	-318.2	
			172			0.029	0.05	182.8	
			173	532		-0.017	0.04	-247.1	
			174			-0.024	0.05	-220.8	
CBR (corrected) at 0.1-inch penetration			514	N/A	169	11.1	5.36	48.3	
					170	7.6	3.97	52.3	
					171	520	32.3	10.28	31.9
					172		31.5	11.22	35.7
					173	536	38.2	12.01	31.5
					174		35.3	12.35	35.0
CBR (corrected) at 0.2-inch penetration			514	N/A	169	15.7	6.64	42.2	
					170	10.3	4.74	45.8	
					171	520	38.5	8.60	22.3
					172		41.7	10.65	25.5
					173	534	46.1	9.46	20.5
					174		46.8	11.65	24.9
Soil Resistance R-Value									
Resistance R-Value of Compacted Soils at 300 psi Exudation Pressure	T190	D2844	169	127	N/A	71.7	3.73	5.2	
			170			70.2	5.19	7.4	
			171	132		70.5	5.30	7.5	
			172			70.2	5.42	7.7	
			173	127		71.7	3.73	5.2	
			174			70.2	5.19	7.4	

Notes: 1. Data provided with Gannett Fleming, Inc. Proficiency Sample evaluation results, 2016

4.8.2 Laboratory Test Methods – PTM, AASHTO, AND ASTM

Laboratory testing must be performed in accordance with a standard test method. In many cases, there are numerous test methods for individual tests, including: Pennsylvania Test Method (PTM), AASHTO Test Method, and ASTM Test Method. Where more than one standard is available for a test, PennDOT Test Methods (PTMs) take precedence and must be used. Current PTMs can be found in [Publication 19](#).

AASHTO Test Methods must be used for all other tests, but if an AASHTO Test Method does not exist, ASTM Test Methods must be used. For the rare cases where a PTM, AASHTO or ASTM test method does not exist, the PGM should recommend a test method to the DGE and obtain approval from the DGE prior to performing the test. The DGE should notify the CGE so that consideration can be given as to whether it is necessary or appropriate to adopt or create a standard test method.

4.8.3 Water Testing

Chemical testing of water should be performed when structures and/or foundations are in contact with bodies of water (e.g., streams, rivers, ponds, lakes, etc.) to determine if the water is corrosive to concrete and/or steel. For example:

- bridge substructures located in the water or within the floodplain of the stream/river/etc.
- retaining walls located along a body of water or within a floodplain
- box and arch culverts (large enough to have S-#)

Chemical testing of groundwater may be prudent or necessary in some situations. In addition, chemical testing of soil samples above and/or below the groundwater may be required for identifying potentially corrosive environments.

4.8.4 Summary of Geotechnical Laboratory Tests

Laboratory tests can be classified as either index tests or performance tests. Index tests provide information that can be used to assess or estimate the general engineering behavior of the soil or rock. For example, index tests can indicate whether a soil is cohesionless or cohesive, or if it is susceptible to consolidation settlement. Soil index tests do not directly measure strength or compressibility, but they can be used in conjunction with correlations to estimate strength and compressibility. Index tests include: natural moisture content, gradation, liquid and plastic limits, unit weight, specific gravity, organic content, chemical analyses, point load strength and slake durability. Performance tests provide a direct measurement of engineering properties, including shear strength, consolidation and permeability. These tests include direct shear, triaxial shear, soil unconfined compressive strength, consolidation, permeability and rock unconfined compressive strength. Index tests are often performed before or in conjunction with performance tests, and index tests are generally less costly to perform than performance tests. Refer to [Chapter 10](#) for Acid Bearing Rock tests and sample size requirements for Fizz Rating, Neutralization Potential, and Total Sulfur rock tests. In addition to total sulfur, there may be situations, for example, with expansive soil formations, where testing for the form of sulfur present, may be important or necessary. A detailed summary of the most commonly performed index and performance tests is provided below.

4.8.4.1 Natural Water (Moisture) Content of Soil

Purpose: To estimate the natural (in-situ) water content of soil.

Uses: Compare the in-situ water content to the liquid and plastic moisture contents (i.e., Atterberg limits). Also, compare the optimum moisture content from moisture-density testing to the in-situ moisture content. This will help determine the effort needed for compaction (i.e., will drying or wetting of soil be required for compaction, or is soil naturally at optimum water content).

Soil Types: All

Soil Sample Types: Disturbed (jars and bags) and undisturbed. Samples must be in an air tight jar/bag/tube to retain the natural water content.

AMRL Certification Required: Yes

AASHTO Test Method:

T 265 – “Standard Method of Test for Laboratory Determination of Moisture Content of Soils”.

ASTM Test Method:

D 2216 – “Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass”.

Minimum Required Sample Size (Note one jar \approx 300 grams):

Max. particle size of No. 40 (0.425 mm):	100 grams
Max. particle size of 12.5 mm (0.5-inch):	300 grams
Max. particle size of 25 mm (1-inch):	500 grams
Max. particle size of 50 mm (2-inch):	1,000 grams

Note actual quantity is dependent upon the percent of the material’s max. particle size present in the sample. Therefore, actual sample sizes that are specified to be sent in to the laboratory will be larger than the amount of material required for the test.

Approximate Cost: Ranges from \$4 to \$15 with an average of \$10 (2016)

Companion Testing (R=Required, S=Suggested): None.

4.8.4.2 Sieve/Gradation Analysis of Soil

Purpose: To estimate the distribution/gradation of particle sizes in soils. Sieve analyses provide gradation of only gravel and sand particles (i.e., larger than No. 200 sieve). Gradation of fines (silt and clay) is determined by hydrometer analysis discussed below.

Uses: Required for laboratory classification of soils. Provide verification of field description and detailed breakdown of particle sizes.

Soil Types: All

Soil Sample Types: Disturbed (jars and bags) and undisturbed.

AMRL Certification Required: Yes

AASHTO Test Method:

T 88 “Standard Method of Test for Particle Size Analysis of Soils”.

ASTM Test Method:

D 422 “Standard Test Method for Particle-Size Analysis of Soils”.

Minimum Required Sample Size (Note one jar \approx 300 grams):

Max. particle size of 9.5 mm (0.375-inch):	500 grams
Max. particle size of 25 mm (1-inch):	2,000 grams
Max. particle size of 50 mm (2-inch):	4,000 grams
Max. particle size of 75 mm (3-inch):	5,000 grams (11 lbs.)

Note actual quantity is dependent upon the percent of the material’s max. particle size present in the sample. Therefore, actual sample sizes that are specified to be sent to the laboratory will be larger than the amount of material required for the test.

Approximate Cost: Ranges from \$80 to \$155 with an average of \$110 (2016)

Companion Testing (R=Required, S=Suggested): Natural water content (S), Hydrometer (S) - particularly when fines content is greater than approximately 30%.

4.8.4.3 Hydrometer Analysis

Purpose: To estimate the distribution/gradation of silt and clay particles (i.e., finer than No. 200 sieve). Note that this test is not needed to classify soils (USCS or AASHTO).

Uses: When combined with sieve analysis it provides a complete gradation of the soil. Provides D_{50} value for scour calculations for fine-grained soils. Needed when geotextile or aggregate filter design is required. Used for design of subgrade stabilization and ground improvement techniques when cement, flyash, etc. are proposed to be mixed with soil. Note that particle size analysis results of very fine grained (clay) soil particles exhibit high variability in the AASHTO PSP program as discussed in [Section 4.8.1](#).

Soil Types: Soils containing approximately 25% or more fines (i.e., silt and clay).

Soil Sample Types: Disturbed (jars and bags) and undisturbed.

AMRL Certification Required: Yes

AASHTO Test Method:

T 88 “Standard Method of Test for Particle Size Analysis of Soils”.

ASTM Test Method:

D 422 “Standard Test Method for Particle-Size Analysis of Soils”.

Minimum Required Sample Size (Note one jar \approx 300 grams): Approximately 100 grams of material passing the No. 10 (2 mm) sieve. Actual quantity is dependent upon the percent of No. 10 material present in the sample. Therefore, actual sample sizes to be sent in to the laboratory will be larger than the amount of material required for the test.

Approximate Cost: See requirements for Sieve/Gradation Analysis. The cost includes Hydrometer Analysis.

Companion Testing (R=Required, S=Suggested): Natural water content (S), Sieve/Gradation (S) – particularly when gravel/sand content is greater than approximately 30%., Liquid and Plastic Limits (S)

4.8.4.4 Liquid and Plastic Limits (Atterberg Limits)

Purpose: To estimate the liquid limit, plastic limit, and plasticity index of soil.

Uses: Required, in addition to sieve analysis, to classify soil (AASHTO and USCS). Limits can also be compared with in-situ moisture content.

Soil Types: All soils except clean sands and gravels (i.e., less than approximately 5% passing No. 200 sieve).

Soil Sample Types: Disturbed (jars and bags) and undisturbed.

AMRL Certification Required: Yes

AASHTO Standard Test Method:

T 89 “Standard Method of Test for Determining the Liquid Limit of Soils”.

T 90 “Standard Method of Test for Determining the Plastic Limit and Plasticity Index of Soils”.

ASTM Test Method:

D 4318 “Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils”.

Minimum Required Sample Size (Note one jar \approx 300 grams): 110 grams of material passing the No. 40 (0.425 mm) sieve. Actual quantity is dependent upon the percent of No. 40 material present in the sample. Therefore, actual sample sizes to be sent in to the laboratory will be larger than the amount of material required for the test.

Approximate Cost: Ranges from \$52 to \$100 with an average of \$75 (2016)

Companion Testing (R=Required, S=Suggested): Natural water content (S), Sieve/Gradation (S), Hydrometer (S)

4.8.4.5 Laboratory Classification of Soil (AASHTO and USCS)

Purpose: To classify soil in accordance with a standardized system.

Uses: Soil classification along with other field and laboratory test results can be used to estimate engineering parameters of soil.

Soil Types: All

Soil Sample Types: Disturbed (jars and bags) and undisturbed.

AMRL Certification Required: Yes (D2487 only)

AASHTO Test Method:

M 145 “Standard Method of Test for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes”. There is currently no AMRL certification required for this test.

ASTM Test Methods:

D 3282 “Standard Practice for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes (AASHTO)”. There is currently no AMRL certification required for this test.

D 2487 “Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)”.

Minimum Required Sample Size (Note one jar \approx 300 grams): See requirements for Sieve/Gradation Analysis and Liquid and Plastic Limits.

Approximate Cost: Ranges from \$210 to \$265 with an average of \$225 (2016)

Companion Testing (R=Required, S=Suggested): Sieve/Gradation (R), Atterberg Limits (R), Natural water content (S), Hydrometer (S) - particularly when fines content is greater than approximately 30%.

4.8.4.6 Density (Unit Weight) of Intact Soil Sample

Purpose: To estimate the in-situ total unit weight or density of the soil sample. Also, can determine in-situ dry unit weight if natural water content testing is performed.

Uses: Use directly in calculations where unit weight of soil is needed.

Soil Types: Soils with adequate cohesion to maintain form when removed from sampler and transported to laboratory.

Soil Sample Types: Undisturbed samples. Also, can use split-spoon samples and hand excavated (block) samples if they maintain their form. Samples must be in an air tight container to retain the natural water content of the soil. Not appropriate for non-plastic silts, clean sands and gravels, or sands and gravels with a non-plastic matrix.

AMRL Certification Required: There is currently no AMRL certification required.

ASTM Test Method:

D 7263 “Standard Test Methods for Laboratory Determination of Density (Unit Weight) of Soil Specimens”.

Minimum Required Sample Size (Note one jar \approx 300 grams): One jar with piece of sample from split-barrel sampler, part of undisturbed sample (Shelby tube), or a block/cube of reasonable size (say three cubic inches minimum).

Approximate Cost: Ranges from \$50 to \$60 with an average of \$55 (2016)

Companion Testing (R=Required, S=Suggested): None.

4.8.4.7 Specific Gravity

Purpose: To estimate specific gravity of soil solids.

Uses: Calculation of weight-volume relationships, including void ratio for consolidation settlement calculations. Note that specific gravity values of soil solids typically have little variation; therefore, specific gravity testing is generally not performed. However, the specific gravity of peat, highly organic soil, and soil with unusual solid minerals can vary more considerably and testing may be prudent.

Soil Types: Fine gravel (maximum particle size of 4.75 mm), sand, silt and clay. Coarse gravel can be tested but not typically useful for geotechnical purposes.

Soil Sample Types: Disturbed (jars and bags) and undisturbed.

AMRL Certification Required: Yes

AASHTO Test Methods:

T 100 “Standard Method of Test for Specific Gravity of Soils” (for maximum particle size of 4.75 mm (No. 4 sieve)).

T 85 “Standard Method of Test for Specific Gravity and Absorption of Coarse Aggregate” (for particles larger than 4.75 mm (No. 4 sieve)).

ASTM Test Method:

D 854 “Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer” (for maximum particle size of 4.75 mm (No. 4 sieve)).

C 127 “Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate”.

Minimum Required Sample Size (Note one jar \approx 300 grams): 100 grams (maximum particle size of 4.75 mm (No. 4 sieve)), of which 25 grams of material should be passing the No. 10 (2 mm) sieve. Actual quantity is dependent upon the percent of No. 10 material present in the sample. Therefore, actual sample sizes to be sent in to the laboratory will be larger than the amount of material required for the test.

Approximate Cost: Ranges from \$55 to \$65 with an average of \$60 (2016)

Companion Testing (R=Required, S=Suggested): None.

4.8.4.8 Organic Content

Purpose: To estimate the organic content of soil.

Uses: Confirm field identification of organic soils. Quantify the amount/percent of organic material in the soil sample.

Soil Types: Peat, organic muck, and soils containing: vegetative matter that has not decayed/decomposed; fresh plant materials such as wood, roots, or grass; or carbonaceous materials such as coal. Samples must be in an air tight container to retain the natural water content of the soil.

Soil Sample Types: Disturbed (jars and bags) and undisturbed.

AMRL Certification Required: Yes (T 267 and D 2974 only)

AASHTO Test Method:

T 267 “Standard Method of Test for Determination of Organic Content in Soils by Loss on Ignition”.

T 194 “Standard Method of Test for Determination of Organic Matter in Soils by Wet Combustion” (This method is only used for suitability of a soil for plant growth.) There is currently no AMRL certification required for this test. However, when determining organic content of soils for engineering purposes, AASHTO T 267 must be used.

ASTM Test Method:

D 2974 “Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils”.

Minimum Required Sample Size (Note one jar \approx 300 grams): 100 grams of material passing the No. 10 (2 mm) sieve. Actual quantity is dependent upon the percent of No. 10 material present in the sample. Therefore, actual sample sizes to be sent in to the laboratory will be larger than the amount of material required for the test.

Approximate Cost: \$115 (2015)

Companion Testing (R=Required, S=Suggested): Natural water content (S).

4.8.4.9 Moisture-Density Relationship (Compaction)

Purpose: To estimate the maximum dry weight density, optimum moisture content and relationship between moisture content and density (i.e., compaction curve).

Uses: Verify that field compaction requirements (moisture content and density) are achieved. Estimate in-place unit weight/density of soils placed during construction to prepare remolded samples for laboratory testing, like strength tests and permeability.

Soil Types: Gravel, sand, silt and clay.

Soil Sample Types: Disturbed - Bulk/bag samples.

AMRL Certification Required: Yes

PTM:

106 “The Moisture-Density Relations of Soil (using a 2.5 kg (5.5 lb.) Rammer and a 305 mm (12-inch) Drop)”. This method is referred to as Standard Compaction. Note that PTM 106 includes Methods A and B; Method B should always be used. Also, note it is required that the test results provide a well-balanced moisture-density curve. Therefore, a minimum of five points must be tested with the results having at least two points on either side of optimum moisture with uniform (roughly equal) increases between plotted, moisture-density points (i.e., the individual points forming the moisture-density curve must be distributed across the graph to represent a reasonably, well-fit, smooth curve).

AASHTO Test Method:

T 99 “Standard Method of Test for Moisture-Density Relations of Soil using a 2.5-kg (5.5-lb) Rammer and a 305-mm (12-in.) Drop”. This method is referred to as Standard Compaction.

T 180 “Standard Method of Test for Moisture-Density Relations of Soil using a 4.54-kg (10-lb) Rammer and a 457-mm (18-in.) Drop”. This method is referred to as Modified Compaction.

ASTM Test Method:

D 698 “Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft³ (600 kN-m/m³))”. This method is referred to as Standard Compaction.

D 1557 “Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft³ (2,700 kN-m/m³))”. This method is referred to as Modified Compaction.

Minimum Required Sample Size: 50 lbs. Note that per the PTM a minimum of 12 lbs. is needed for the test. The 12 lbs. is only material finer than ¾-inch; therefore, additional material is needed if the sample contains particles larger than ¾-inch. Additionally, the 12 lb. minimum is based on reuse of sample during the test. As discussed in PTM 106, reuse is not recommended where the soil material is fragile or where the soil is heavy-textured clayey material that is difficult to incorporate water. Consequently, when possible a 50 lb. sample shall be obtained to account for particles greater than ¾-inch and to eliminate the need to reuse sample for the test.

Approximate Cost (includes 3 molds): Ranges from \$128 to \$230 with an average of \$190 (2016)

Companion Testing (R=Required, S=Suggested): Sieve/Gradation (R), Natural water content (S), Atterberg Limits (S), Hydrometer (S) - particularly when fines content is greater than approximately 30%.

Information Needed by Laboratory (supplied by customer):

Specify Method B of PTM 106.

4.8.4.10 California Bearing Ratio (CBR)

Purpose: To estimate the California Bearing Ratio (CBR) of soil.

Uses: Required for pavement design when subgrade materials are known. Note that CBR test results exhibit high variability in the AASHTO PSP program as discussed in [Section 4.8.1](#).

Soil Types: Gravel, sand, silt and clay.

Soil Sample Types: Disturbed - Bulk/bag samples.

AMRL Certification Required: Yes

AASHTO Test Method:

T 193 “Standard Method of Test for the California Bearing Ratio”. Note that for a given sample, three 1-point tests compacted to the maximum dry density and optimum moisture content obtained using PTM 106, Method B are required (See Note 6 in AASHTO T 193), as opposed to one 3-point test. The results of the three 1-point tests should be averaged and checked for consistency to remove any outliers.

ASTM Test Method:

D 1883 “Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils”.

Minimum Required Sample Size: 50 lbs. (100 lbs. preferred) for three 1-point molds on fine-grained material (i.e., all material passing the 3/8-inch sieve molded to optimum moisture/maximum dry density). 300 lbs. (400 lbs. preferred) for three 1-point molds on coarse-grained material (i.e., some material retained on the 3/8-inch sieve molded to optimum moisture/maximum dry density)

Approximate Cost (includes three 1-point molds): Ranges from \$325 to \$675 with an average of \$535 (2016)

Companion Testing (R=Required, S=Suggested): Compaction (R), Sieve/Gradation (R), Natural water content (S), Atterberg Limits (S), Hydrometer (S) - particularly when fines content is greater than approximately 30%.

Information Needed by Laboratory (supplied by customer):

1. Specify water content or range of water content and dry density. Normally maximum dry density and optimum moisture content are used.
2. Specify special surcharge weights if different than the test minimum of 10 lbs. The weight is intended to represent the load from the pavement section (i.e., subbase, base course and wearing course).

4.8.4.11 Consolidation

Purpose: To estimate consolidation settlement properties of soil.

Uses: Provides information necessary to estimate/predict magnitude and time rate of consolidation settlement.

Soil Types: Saturated cohesive soils.

Soil Sample Types: Typically undisturbed; remolded samples are rarely used.

AMRL Certification Required: Yes

AASHTO Test Method:

T 216 “Standard Method of Test for One-Dimensional Consolidation Properties of Soils”.

ASTM Test Method:

D 2435 “Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading”.

Minimum Required Sample Size: 4-inch tube sample (assume one Shelby tube).

Approximate Cost (includes Shelby tube extrusion cost): Ranges from \$270 to \$800 with an average of \$545 (2016)

Companion Testing (R=Required, S=Suggested): Specific Gravity (R), Shelby tube sample extrusion (R), Sieve/Gradation (S), Atterberg limits (S), Hydrometer (S). Natural water content and unit weight are determined as part of this test.

Information Needed by Laboratory (supplied by customer):

1. Specify the load cycle. Typical cycle is 0.25, 0.5, 1, 2, 4, 8, 16, 32, 8, 2, 0.25 tsf. Some labs may not be capable of applying 32 tsf load, which is acceptable in most cases.
2. Specify Method A or B. Method A is typically used and requires each load be held for 24 hours. Method B allows load duration to be reduced to the time it takes for completion of primary consolidation. This method can be considered for soils with low plasticity index, particularly when test results are needed quickly.
3. If remolded samples are used, specify density and water content.

4.8.4.12 Direct Shear

Purpose: To estimate the drained/effective peak shear strength of soil. See [Section 4.9.4](#) for detailed discussion.

Uses: Provides shear strength information (internal friction angle and cohesion) needed for slope stability analysis, foundation design (bearing resistance, friction and end bearing (soil) piles and drilled shaft, etc.) and retaining wall design.

Soil Types: Typically sands and fine-grained soils. Dependent upon size of testing apparatus. See detailed discussion of direct shear testing.

Soil Sample Types: Disturbed (jars and bags) and undisturbed. Sample type preference is dependent upon design application. See detailed discussion of direct shear testing.

AMRL Certification Required: Yes

AASHTO Test Method:

T 236 “Standard Method of Test for Direct Shear Test of Soils under Consolidated Drained Conditions”. **RESIDUAL VALUES ARE OBTAINED FROM THE RESIDUAL STRENGTH PORTION OF THE STRESS-STRAIN PLOT OF THE DIRECT SHEAR TEST. IT IS NECESSARY TO RUN THE DIRECT SHEAR TEST WITH SUFFICIENT DISPLACEMENT TO ASSURE RESIDUAL SHEAR HAS BEEN OBSERVED.**

ASTM Test Method:

D 3080 “Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions”.

Minimum Required Sample Size (Note one jar \approx 300 grams): Dependent upon size of testing apparatus. See detailed discussion of direct shear testing. Generally, a 6-inch tube sample (assume one Shelby tube) or 1,000 grams. 600 grams of material should be passing the No. 10 (2 mm) sieve. Actual quantity is dependent upon the percent of No. 10 material present in the sample. Therefore, actual sample sizes to be sent in to the laboratory will be larger than the amount of material required for the test.

Approximate Cost (includes required 3 normal loads): Ranges from \$450 to \$795 with an average of \$530 (2016).

Companion Testing (R=Required, S=Suggested): Sieve/Gradation (R), Specific Gravity (R), Shelby tube sample extrusion, remolding sample, or block sample preparation (R), Atterberg limits (S), Hydrometer (S), Compaction (S) if remolded samples are being used. Natural water content and unit weight are determined as part of this test.

Information Needed by Laboratory (supplied by customer):

1. Specify normal loads. Commonly 4, 8, and 12 ksf are used.
2. If remolded samples are used, specify density and water content.

4.8.4.13 Residual Direct Shear

Purpose: To estimate the residual shear strength of soil.

Uses: Provides residual shear strength estimate along weak plane(s) within soil deposit, such as along a slope stability/landslide failure plane (active or inactive) or along thin varves. This test is useful for slope stability analysis of in-situ soils and landslide remediation design.

Soil Types: Cohesive soils (clay, silty clay, and clayey silt).

Soil Sample Types: Disturbed (jars and bags) and undisturbed. Sample type preference is dependent upon design application. See detailed discussion of direct shear testing.

AMRL Certification Required: There is currently no AMRL certification required.

Test Method:

Neither AASHTO nor ASTM has a test method for residual direct shear. The United States Army Corps of Engineers EM 1110-2-1906, Appendix IX A is not approved for use by the Department. **RESIDUAL VALUES ARE OBTAINED FROM THE RESIDUAL STRENGTH PORTION OF THE STRESS-STRAIN PLOT OF THE NORMAL DIRECT SHEAR TEST (T 236). IT IS NECESSARY TO RUN THE DIRECT SHEAR TEST WITH SUFFICIENT DISPLACEMENT TO ASSURE RESIDUAL SHEAR HAS BEEN OBSERVED.**

Approximate Cost: No additional cost. See [Section 4.8.4.12](#) for additional information.

4.8.4.14 Triaxial Compression (Shear) – Unconsolidated, Undrained (UU)

Purpose: To estimate the undrained shear strength (i.e., cohesion) of cohesive soil.

Uses: Provides shear strength parameter for total stress (i.e., short term) stability analysis. Represents field conditions where load is applied rapidly (i.e., embankment construction or loading of a spread footing) and the foundation soil does not have adequate time to drain (i.e., pore water pressure does not dissipate), and therefore does not consolidate.

Soil Types: Cohesive soils (clay, silty clay, and clayey silt).

Soil Sample Types: Typically undisturbed, but can use disturbed (bag).

AMRL Certification Required: Yes

AASHTO Test Method:

T 296 “Standard Method of Test for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression”.

ASTM Test Method:

D 2850 “Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils”.

Minimum Required Sample Size: 24-inch sample (1 tube). Usually two tubes are required since full recovery is not always achieved and some portion of tube may not be suitable for testing (i.e., disturbance at top, piece(s) of gravel, thin layer of soil within the tube not representative of stratum to be tested, etc.). If using disturbed/remolded samples, 25 lbs.

Approximate Cost (includes 3 required confining loads): \$600 (2015).

Companion Testing (R=Required, S=Suggested): Specific gravity (R), Shelby tube sample extrusion, remolding sample, or block sample preparation (R), Sieve/Gradation (S), Atterberg limits (S), Hydrometer (S), Compaction (S) if remolded samples are being used. Natural water content and unit weight are determined as part of this test.

Information Needed by Laboratory (supplied by customer):

1. Specify chamber (consolidation) pressure.
2. If remolded samples are used, specify density and water content.

4.8.4.15 Triaxial Compression (Shear) – Consolidated, Undrained (CU) with or without pore pressure measurement

Purpose: To estimate the total shear strength of cohesive soil. If pore pressure measurements are taken during the test, effective shear strength can also be estimated.

Uses: Provides shear strength parameter for total stress (i.e., short term) stability analysis, similar to the UU test. However, unlike the UU test, the CU test is performed by consolidating the sample prior to applying the shear stress. The CU test can be used to represent conditions in the field where load has been applied and foundation soils have had time to consolidate but excess pore water pressure is still present. It also represents conditions of previous consolidation, like glaciation, but excess pore pressure due to newly applied loads. By also measuring pore pressure during testing, the CU test can be used to represent conditions in the field where load has been applied and foundation soils have consolidated and excess pore water pressure has dissipated (i.e., long-term conditions).

Soil Types: Cohesive soils (clay, silty clay, and clayey silt).

Soil Sample Types: Typically undisturbed, but can use disturbed (bag).

AMRL Certification Required: Yes

AASHTO Test Method:

T 297 “Standard Method of Test for Consolidated, Undrained Triaxial Compression Test on Cohesive Soils”.

ASTM Test Method:

D 4767 “Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils”.

Minimum Required Sample Size: 24-inch sample (one tube). Usually two tubes required since full recovery is not always achieved and some portion of tube may not be suitable for testing (i.e., disturbance at top, piece(s) of gravel, thin layer of soil within the tube not representative of stratum to be tested, etc.). If using disturbed/remolded samples, 25 lbs.

Approximate Cost (includes 3 required confining loads): \$1,000 (2015) for no pore pressure measurement. \$1,600 (2015) for pore pressure measurement.

Companion Testing (R=Required, S=Suggested): Specific gravity (R), Shelby tube sample extrusion, remolding sample, or block sample preparation (R), Sieve/Gradation (S), Atterberg limits (S), Hydrometer (S), Compaction (S) if remolded samples are being used. Natural water content and unit weight are determined as part of this test.

Information Needed by Laboratory (supplied by customer):

1. Specify chamber (consolidation) pressure.
2. If remolded samples are used, specify density and water content.

4.8.4.16 Triaxial Compression (Shear) – Consolidated, Drained (CD)

Purpose: To estimate the effective shear strength of cohesive soil.

Uses: This test is performed by allowing drainage (i.e., no excess pore pressure) and consolidation while the shearing stress is applied. Since cohesive soil generally drains very slowly, the shear stress must be applied at a very low rate. Consequently, the test takes considerable time to conduct, is costly and is therefore rarely performed. In lieu of this test, the triaxial CU test with pore water pressure measurements is typically performed to estimate effective (i.e., drained or long term strength) shear strength.

Soil Types: Cohesive soils (clay, silty clay, and clayey silt).

Soil Sample Types: Typically undisturbed, but can use disturbed (bag).

AMRL Certification Required: There is currently no AMRL certification required.

Test Method:

Neither AASHTO nor ASTM has a test method for CD triaxial testing. The United States Army Corps of Engineers EM 1110-2-1906, Appendix X can be used.

Minimum Required Sample Size: 24-inch sample (one tube). Usually two tubes required since full recovery is not always achieved and some portion of tube may not be suitable for testing (i.e., disturbance at top, piece(s) of gravel, thin layer of soil within the tube not representative of stratum to be tested, etc.). If using disturbed/remolded samples 25 lbs.

Approximate Cost: Laboratories generally provide a price quote based on a project by project basis since this test is rarely performed because it is extremely time consuming and costly. (2015)

Companion Testing (R=Required, S=Suggested): Specific gravity (R), Shelby tube sample extrusion, remolding sample, or block sample preparation (R), Sieve/Gradation (S), Atterberg limits (S), Hydrometer (S), Compaction (S) if remolded samples are being used. Natural water content and unit weight are determined as part of this test.

Information Needed by Laboratory (supplied by customer):

1. Specify chamber (consolidation) pressure.
2. If remolded samples are used, specify density and water content.

4.8.4.17 Unconfined Compression (Soil)

Purpose: To estimate the unconfined compressive strength of cohesive soil.

Uses: Provides an estimate of the undrained shear strength (i.e., cohesion).

Soil Types: Cohesive soils (clay, silty clay, and clayey silt).

Soil Sample Types: Typically undisturbed, but can use disturbed (bag).

AMRL Certification Required: Yes

AASHTO Test Method:

T 208 “Standard Method of Test for Unconfined Compressive Strength of Cohesive Soil”.

ASTM Test Method:

D 2166 “Standard Test Method for Unconfined Compressive Strength of Cohesive Soil”.

Minimum Required Sample Size: Six-inch piece of tube sample per test (typically, three samples). If using disturbed/remolded sample, 10 lbs.

Approximate Cost (per test): \$110 (2015)

Companion Testing (R=Required, S=Suggested): Shelby tube sample extrusion, remolding sample, or block sample preparation (R), Sieve/Gradation (S), Atterberg limits (S), Hydrometer (S). Natural water content and unit weight are determined as part of this test.

Information Needed by Laboratory (supplied by customer):

If remolded samples are used specify density and water content.

4.8.4.18 Unconfined Compression (Rock)

Purpose: To estimate the unconfined compressive strength of bedrock.

Uses: Provides estimate of bedrock strength needed for design of spread footings, drilled shafts, micropiles, rock anchors, etc. Provides an indication of the ease/difficulty and equipment needed to excavate bedrock for roadway cuts, spread footings, drilled shafts, etc. This test is preferred over Point Load tests if required sample size is available.

Rock Types: All

Rock Sample Types: Cylindrical Core

AMRL Certification Required: Yes

ASTM Test Method:

D 7012 “Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens”, Method C.

Minimum Required Sample Size (per test/break): 1.875-inch minimum diameter and length to diameter ratio of 2 to 2.5. Includes NW and HW conventional rock core, and NX, NQ, HQ and HQ₃ wireline rock core.

Approximate Cost (per break): \$40 (2015)

Companion Testing (R=Required, S=Suggested): None.

4.8.4.19 Point Load

Purpose: To estimate point load strength index of bedrock.

Uses: Provides point load strength index of bedrock which allows calculation of unconfined compressive strength. This test is not a direct measure of unconfined compression test, and should typically only be used in place of D 7012 if the sample size requirement of D 7012 cannot be met.

Rock Types: All.

Rock Sample Types: Cores (preferred), blocks or irregular lumps.

AMRL Certification Required: Yes

ASTM Test Method:

D 5731, “Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications”.

Minimum Required Sample Size (per break): For core a diameter of 1.2 to 3.3 inches, but +/- 2-inch is preferred, and length greater than 0.3 times the diameter. See specification for details on size requirements for core (diametral and axial), block and irregular samples. A minimum of 10 samples is required for core or block tests, and a minimum of 20 irregular lump samples is required.

Approximate Cost (per break): \$20 (2015)

Companion Testing (R=Required, S=Suggested): None.

4.8.4.20 Slake Durability

Purpose: To estimate the slake durability index of shale or other similar bedrock.

Uses: Provides a qualitative indication to the slaking potential of bedrock for design of foundations bearing on rock (spread footing, drilled shafts, piles, etc.) and rock cuts. There is no accepted durability index reference value for various rocks; therefore, the test results must be used with engineering judgment, past experience, etc.

Rock Types: Slake prone rocks – shale, claystone, etc.

Rock Sample Types: Rock cores or rock blocks

AMRL Certification Required: Yes

ASTM Test Method:

D 4644 “Standard Test Method for Slake Durability of Shales and Similar Weak Rocks”.

Minimum Required Sample Size: Ten, roughly equi-dimensional fragments weighing 40 to 60 grams each (total sample weight 450 to 550 g, about 1 lb.). Retain natural moisture of samples prior to testing.

Approximate Cost: \$250 (2015)

Companion Testing (R=Required, S=Suggested): None.

4.8.4.21 Resistivity (Soil)

Purpose: To estimate the resistivity of soil in the laboratory.

Uses: Provides an indication of the corrosion potential of soils to steel, typically with respect to foundation design (i.e., steel piles and concrete reinforcement).

Soil Types: All, but only coarse sand (2 mm) and finer particles are used in test.

Soil Sample Types: Disturbed (jars and bags) and undisturbed.

AMRL Certification Required: Yes

AASHTO Test Method:

T 288 “Standard Method of Test for Determining Minimum Laboratory Soil Resistivity”.

Minimum Required Sample Size (Note one jar \approx 300 grams): 1,500 grams (max. particle size of 2 mm).

Approximate Cost: Ranges from \$40 to \$180 with an average of \$125 (2016)

Companion Testing (R=Required, S=Suggested): pH (R), Sulfate (S), Chloride (S)

4.8.4.22 pH (Soil)

Purpose: To estimate the pH of soil.

Uses: Provides an indication of the corrosion potential of soils to steel, typically with respect to foundation design (i.e., steel piles and concrete reinforcement).

Soil Types: All, but only coarse sand (2 mm) and finer particles are used in test.

Soil Sample Types: Disturbed (jars and bags) and undisturbed.

AMRL Certification Required: Yes

AASHTO Test Method:

T 289 “Standard Method of Test for Determining pH of Soil for Use in Corrosion Testing”.

Minimum Required Sample Size (Note one jar \approx 300 grams): 100 grams (max. particle size of 2 mm).

Approximate Cost: \$35 (2015)

Companion Testing (R=Required, S=Suggested): Resistivity (R), Sulfate (S), Chloride (S)

4.8.4.23 Sulfate Ion (Soil)

Purpose: To estimate water-soluble sulfate in soil.

Uses: Provides an indication of the corrosion potential of soil on concrete (footings, piles, drilled shafts, etc.).

Soil Types: All, but only coarse sand (2 mm) and finer particles are used in test.

Soil Sample Types: Disturbed (jars and bags) and undisturbed.

AMRL Certification Required: There is currently no AMRL certification required.

AASHTO Test Method:

T 290 “Standard Method of Test for Determining Water-Soluble Sulfate Ion Content in Soil”.

ASTM Test Method:

C 1580 “Standard Test Method for Water-Soluble Sulfate in Soil”.

Minimum Required Sample Size (Note one jar ≈ 300 grams): 250 grams (max. particle size of 2 mm).

Approximate Cost: \$35 (2015)

Companion Testing (R=Required, S=Suggested): Resistivity (R), pH (R), Chloride (R)

4.8.4.24 Chloride Ion (Soil)

Purpose: To estimate water-soluble chloride in soil.

Uses: Provides an indication of the corrosion potential of soil on concrete (footings, piles, drilled shafts, etc.).

Soil Types: All, but only coarse sand (2 mm) and finer particles are used in test.

Soil Sample Types: Disturbed (jars and bags) and undisturbed.

AMRL Certification Required: There is currently no AMRL certification required.

AASHTO Test Method:

T 291 “Standard Method of Test for Determining Water-Soluble Chloride Ion Content in Soil”.

Minimum Required Sample Size (Note one jar ≈ 300 grams): 250 grams (max. particle size of 2 mm).

Approximate Cost: \$35 (2015)

Companion Testing (R=Required, S=Suggested): Resistivity (R), pH (R), Sulfate (R)

4.8.4.25 Hydraulic Conductivity (Permeability) (Soil)

Purpose: To estimate the soil’s ability to transmit water through pore spaces or fractures when submitted to a hydraulic gradient.

Uses: Provides an indication of the permeability of water through soil.

Soil Types: All, refer to ASTM D 2434 for high permeability soils.

Soil Sample Types: Disturbed (jars and bags) and undisturbed.

AMRL Certification Required: Yes

ASTM Test Method:

D 5084 “Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter”.

Minimum Required Sample Size: Determined per method, soil type, and/or testing laboratory.

Approximate Cost: \$540 (2015)

Companion Testing (R=Required, S=Suggested): pH (S), Sulfate (S), Chloride (S)

Information Needed by Laboratory (supplied by customer):

If remolded samples are used specify density and water content.

4.8.4.26 Conductivity/Resistivity (Water)

Purpose: To estimate conductivity/resistivity of water. Resistivity is the inverse of conductivity.

Uses: Provides an indication of the corrosion potential of water on steel, typically with respect to foundation design (i.e., steel piles and concrete reinforcement).

Water Sample Types: Streams/Creeks/Rivers/etc., Ponds/Lakes/etc., and ground water.

AMRL Certification Required: There is currently no AMRL certification required.

ASTM Test Method:

D 1125 “Standard Test Methods for Electrical Conductivity and Resistivity of Water” (Method A). Note that results from this test method are reported in Siemens/cm. Conversion to ohm-cm is completed by taking the inverse of the Siemens value (i.e., 0.00005 siemens/cm equals 20,000 ohm-cm).

Minimum Required Sample Size: Determined by testing agency.

Approximate Cost: \$35 (2015)

Companion Testing (R=Required, S=Suggested): pH (R), Sulfate (S), Chloride (S)

4.8.4.27 pH (Water)

Purpose: To estimate pH of water.

Uses: Provides an indication of the corrosion potential of water to steel, typically with respect to foundation design (i.e., steel piles and concrete reinforcement).

Water Sample Types: Streams/Creeks/Rivers/etc., Ponds/Lakes/etc., and ground water.

AMRL Certification Required: There is currently no AMRL certification required.

ASTM Test Method:

D 1293 “Standard Test Methods for pH of Water”.

Minimum Required Sample Size: Determined by testing agency.

Approximate Cost: \$35 (2015)

Companion Testing (R=Required, S=Suggested): Resistivity (R), Sulfate (S), Chloride (S)

4.8.4.28 Sulfate (Water)

Purpose: To estimate water-soluble chloride in soil.

Uses: Provides an indication of the corrosion potential of water on concrete (footings, piles, drilled shafts, etc.).

Water Sample Types: Streams/Creeks/Rivers/etc., Ponds/Lakes/etc., and ground water.

AMRL Certification Required: There is currently no AMRL certification required.

ASTM Test Method:

D 516 “Standard Test Method for Sulfate Ion in Water”.

Minimum Required Sample Size: Determined by testing agency.

Approximate Cost: \$35 (2015)

Companion Testing (R=Required, S=Suggested): Resistivity (R), pH (R), Sulfate (R)

4.8.4.29 Chloride (Water)

Purpose: To estimate water-soluble chloride in water.

Uses: Provides an indication of the corrosion potential of water on concrete (footings, piles, drilled shafts, etc.).

Water Sample Types: Streams/Creeks/Rivers/etc., Ponds/Lakes/etc., and ground water.

AMRL Certification Required: There is currently no AMRL certification required.

ASTM Test Method:

D 512 “Standard Test Method for Chloride Ion in Water”.

Minimum Required Sample Size: Determined by testing agency.

Approximate Cost: \$35 (2015)

Companion Testing (R=Required, S=Suggested): Resistivity (R), pH (R), Sulfate (R)

4.9 GEOTECHNICAL TEST CONSIDERATIONS

This section discusses issues concerning some of the more complex laboratory tests involving soils. It focuses on considerations involving strength testing, consolidation, and moisture density relationships.

4.9.1 Moisture Density, PTM 106

The Moisture Density Test, which is often referred to as a Proctor Compaction Test, is performed to establish a relationship between water content and unit weight of the soil (i.e., compaction curve). Ultimately, the test yields the maximum dry density (i.e., unit weight) and the water content (i.e., optimum water content) at which this maximum density occurs.

The Moisture Density test is performed for several reasons. Commonly, the test is performed for construction quality control purposes. The values obtained from the laboratory tests are compared with density and water content values measured during construction with a nuclear density gauge or sand cone to ensure the project specifications are met. The test is also performed to support other laboratory tests, including California Bearing Ratio (CBR), or when remolded samples are used for testing (i.e., strength testing and permeability). The Moisture Density test provides an estimate of the moisture and density that the soil will be compacted to during construction, and these values can be used to remold samples for laboratory testing which simulate field conditions.

The compaction effort used on the soil in the laboratory test is intended to simulate the compaction effort used on the soil in the field during construction. PTM 106, which is a slight modification to AASHTO T 99 (Standard Compaction), is typically used to simulate field compaction for embankment, subgrade and foundation design and construction. However, in some cases it may be necessary to require a higher compactive effort during construction. In these situations consideration should be given to performing the laboratory testing in accordance with AASHTO T 180, which is referred to as the Modified Compaction test.

4.9.2 California Bearing Ratio AASHTO, T 193

A key input for the design of bituminous and concrete pavement is the subgrade strength (i.e., resilient modulus, M_r). The resilient modulus value used has a critical effect on the design of economical, durable pavement sections; therefore, it is imperative that soil subgrade samples be collected appropriately and tested properly so that a representative resilient modulus value is

used. Guidance is provided below for the sampling and testing of subgrade soil for pavement design.

In accordance with Publication 242, the resilient modulus value for Department projects is estimated through a correlation with the California Bearing Ratio (CBR). The CBR is determined in accordance with laboratory test AASHTO T 193-10, and the resilient modulus is calculated by multiplying the CBR by 1,500. Note that at this time the laboratory test method for determining the resilient modulus directly (i.e., AASHTO T 307) cannot be used on Department projects per Publication 242.

Soil samples for CBR testing should be collected on all projects where pavement design is required, unless in-situ test methods for determining resilient modulus are approved for use (e.g., dynamic cone penetrometer (DCP) or falling weight deflectometer (FWD)). Where new pavement sections will simply match existing sections, pavement design is not needed, and CBR testing does not need to be performed. Examples would include some widening projects or small projects (i.e., less than 500 feet of continuous paving on divided highways, less than 1,000 feet of continuous paving on undivided highways, and less than 1,000 feet of total roadway approach paving for bridges).

Per the AASHTO test method 75 pounds of material is needed to perform the CBR test. However, if the soil sample has oversize particles (i.e., material larger than the $\frac{3}{4}$ -inch sieve), obtain an additional sample so that the oversize material can be replaced with finer grained material per the test method. Refer to [Section 4.8.4.10](#) for required minimum samples sizes for fine-grained and coarse-grained materials.

Similar to most geotechnical testing, the appropriate number of CBR tests performed is dependent upon several factors, including the size of the project and the variability of the subgrade materials. Below are some guidelines to help determine the appropriate number of tests for a project; however, each project is unique and must be considered individually:

- A minimum of one test per subgrade soil type should be performed on projects for local and collector roads with less than approximately 2,500 feet of paving.
- A minimum of three tests per subgrade soil type should be performed on projects for local and collector roads with more than approximately 2,500 feet of paving
- A minimum of three tests per subgrade soil type should be performed on projects for interstate and arterial roads.
- Two tests per subgrade soil type are generally not recommended because if the test results vary considerably, it is difficult to determine which test is representative. If multiple tests are performed, a minimum of three is recommended.
- Fine-grained soils generally will have a lower CBR value compared to coarse-grained soils; therefore, testing should be performed on fine-grained soils when encountered, unless they are in an isolated area and will be treated/stabilized. When both fine and coarse-grained subgrade soils are encountered on a project, it may not be necessary to test the coarse-grained material, assuming a single CBR value will be used to design the pavement for the entire project. However, coarse-

grained soils containing fines (i.e., sm, sc, gm, gc, etc.) should be tested since the fine-grained portion of these soils often controls the behavior of the soil.

As per AASHTO T 193-10, the test method is primarily intended for, but not limited to, evaluating the strength of cohesive materials having maximum particle sizes less than ¾-inch. Gradation of materials having maximum particle sizes greater than ¾-inch is modified so that the material used for tests passes the ¾-inch sieve. The percentage by weight of materials retained on the ¾-inch sieve are replaced with material passing the ¾-inch sieve and retained on the No. 4 sieve. CBR results for materials having substantial percentages of particles retained on the No. 4 sieve are more variable than for finer materials. Because of this, it is recommended to consider performing additional tests when the materials encountered are anticipated to contain significant percentages of particles retained on the No. 4 sieve.

Although AASHTO T 193-10 allows for testing material in an unsoaked condition, all samples for Department projects must be soaked prior to penetration unless otherwise directed by the Department. During soaking a surcharge weight (i.e., 10 pound weight, minimum) is applied to simulate the intensity of loading from the pavement. Based on the equation below, a 10 -pound weight is equivalent to approximately 4-inches of pavement. Additional weight may be applied in increments of five (5) pounds for thicker pavement if approved by the District Pavement Engineer. Note that the CBR value for the same soil will likely increase as the surcharge weight is increased.

To use a surcharge weight greater than the 10-pound minimum, the pavement design engineer must estimate the thickness of the proposed pavement prior to laboratory testing, and must direct the laboratory accordingly. The surcharge weight for the proposed pavement can be estimated using the following equation:

$$W_s = (h_p \gamma_p) A_m$$

where:

W_s = weight of surcharge (lbs)

h_p = height of bituminous or plain cement concrete pavement (not including subbase) (ft)

γ_p = unit weight of pavement (lbs/ft³)

A_m = area of CBR mold (ft²)

It is generally recommended to only consider the bituminous or cement concrete portion of the pavement section, and not the subbase, when calculating the surcharge weight. Including the subbase in the surcharge weight calculation can result in very high, and possibly unrealistic, surcharge weights. The typical pavement (bituminous or cement) has a unit weight of approximately 150 pounds per cubic foot, and the area of a 6-inch diameter CBR mold is approximately 0.2 square feet.

After the samples have soaked, they are penetrated with a piston, and a plot of load versus penetration is constructed from the test results. The CBR is calculated in percent by dividing the load required to penetrate the samples 0.1 and 0.2 inch by the standard load of 1,000 and 1,500 psi, respectively. The CBR is generally selected at 0.10-inch penetration. However, if

the CBR at 0.20-inch penetration is greater, the test must be rerun. It is not acceptable to simply use the CBR at 0.20-inch penetration. If the results of the rerun test are similar to the original test, and the CBR at 0.20-inch penetration is greater than at 0.10-inch, the CBR at 0.20-inch penetration should be used.

For projects where several CBR tests are performed on similar soil types, judgment must be used to determine the appropriate CBR value to use for pavement design. It is generally not appropriate to use the average CBR value unless all of the values are relatively close. If there is a high or low value that appears to be an anomaly, it may be appropriate to disregard this value and average the remaining similar values to determine the design CBR value. When there is a significant range in the CBR values, using an average value would likely result in an inadequately designed pavement section or an excessively thick pavement section over the majority of the project alignment. In most cases, it is prudent to use a value toward the low end of the range of test results. If the lowest value appears to be an anomaly or is only representative of an isolated area of the project, it may be economical to perform subgrade improvement in the area where the low value was obtained and use a higher CBR value for the pavement design to better model the overall project conditions. Note that per Publication 242, undercutting and/or stabilizing the subgrade may be required where the CBR value is less than five. Additionally, Publication 242 requires that recommended design CBR values greater than 10 be approved by the Chief Geotechnical Engineer. Refer to Publication 242 for additional details regarding determination of an appropriate CBR value for design.

4.9.3 Consolidation, AASHTO T 216

Consolidation testing is performed to estimate consolidation (not immediate) settlement characteristics of fine-grained, typically cohesive, saturated soils. When saturated, cohesive soil is loaded by an embankment, spread footing foundation, etc., the load is initially carried by the water trapped in the void space of a soil mass. This results in excess porewater pressure in the soil. Since the water in the voids is under pressure, the water is forced out of the voids. The time for drainage to occur is dependent upon the permeability of the soil, the thickness of the layer, and the presence of drainage layers (i.e., higher permeability soil like sand or gravel layers, or manmade inclusions like wick or sand drains). Once the excess porewater pressure in the soil mass dissipates, the load is carried fully by the soil particles. This process is referred to as primary consolidation. Since granular soils have high permeability, they drain quickly when loaded and consolidation settlement is not applicable.

Consolidation testing should be performed on cohesive, saturated foundation (in-situ) soils that will be subjected to vertical loads, such as from an embankment or a structure foundation. Since structure foundations underlain by cohesive soil typically utilize deep foundations in lieu of spread footings, consolidation testing may not be needed for structure foundation design. However, where approach embankments or other vertical loads may cause settlement of the soil surrounding a deep foundation, consolidation testing may be needed for foundation design to help estimate if downdrag may occur.

Since this test is typically performed on foundation soil (i.e., in-situ and not a proposed embankment) undisturbed soil samples, such as from a thin-walled sampler (Shelby tube), must

be used to perform the test. Remolded samples cannot be used to estimate in-situ consolidation settlement characteristics. If consolidation properties of a proposed embankment are needed, which is very rare in the transportation industry, a remolded sample would be appropriate to use.

Because soil samples from Shelby tubes are typically used to perform this test, it is important to reiterate the importance of properly obtaining, preserving, transporting and storing undisturbed samples. The consolidation test is one of the most “sensitive” tests with respect to sample quality. Sample disturbance will most likely affect the test results, and the test results may not be representative of the in-situ conditions. It is not necessarily obvious when sample disturbance has affected the test results. An indication of sample disturbance is when there is no distinct break in the slope of the plotted laboratory consolidation curve (void ratio versus log of pressure) between recompression and virgin compression. However, this lack of break in slope is sometimes seen in low plasticity soils so it is not an absolute sign of sample disturbance.

In addition to properly obtaining, preserving, transporting and storing undisturbed samples, it is critical that testing of undisturbed samples be performed as soon as possible after obtaining the sample. In addition to losing natural moisture, soil samples inside Shelby tubes often swell over time. This swelling makes it very difficult to extrude the sample from the tube without significantly disturbing the sample. If the testing cannot be performed within a few weeks of obtaining the sample, the sample should be extruded from the tube and stored in an airtight container to retain the natural water content until it can be tested.

The consolidation test determines several parameters, including: preconsolidation pressure (P_c), virgin compression index (C_c), recompression index (C_r), sometimes referred to as swelling or rebound index (C_s), and the coefficient of consolidation (c_v). The preconsolidation pressure (P_c) is possibly the most valuable parameter determined by the test. The preconsolidation pressure represents the magnitude of the maximum stress (i.e., effective overburden pressure) that the soil has experienced since it was deposited. The stress history of a soil deposit can be affected by numerous things, including glaciation, erosion, desiccation, groundwater fluctuation, and more. Once the preconsolidation pressure is known, it can be compared to the present day effective overburden pressure to determine if the soil is normally consolidated or overconsolidated. The magnitude of consolidation settlement can be estimated by using the appropriate equation and the C_c and/or C_r values obtained from the test results, and the time rate of settlement can be estimated using the coefficient of consolidation (c_v).

4.9.4 Direct Shear, AASHTO T 236

The direct shear test is performed to estimate the peak shear strength of soil. This test is performed slowly to ensure that the soil sample is consolidated and drained (i.e., no excess pore water pressure) during the test. This test is intended to represent the long term strength of the soil. It models soil conditions after construction/loading has taken place and the soil has had a chance to fully consolidate/drain (i.e., no excess pore water pressure). Consequently, this strength state is often referred to as the S (slow) strength. These test results can be used directly in slope stability analysis and foundation design (spread footings bearing on soil, friction piles, lateral earth pressure, etc.).

The direct shear test is performed by placing the soil sample in the direct shear box, which can be either square or circular. The shear box consists of two halves to allow them, and the soil sample, to move horizontally independent of each other and develop a shear/failure plane. A normal (vertical) stress selected by the engineer is applied to the sample. After the soil sample has consolidated/drained due to the applied normal stress, the halves of the box are slowly displaced horizontally from one another causing shearing. Shearing occurs at a constant rate while shear resistance and horizontal displacement are measured. Shearing continues until a decrease in shearing resistance is measured, which signifies a shear failure (i.e., excessive horizontal movement) of the sample.

Typically, three test specimens are sheared at increasing normal stresses to estimate the shear strength of the soil. Mohr-Coulomb failure envelopes can be drawn from the test results by plotting effective normal stress on the x-axis and shear stress on the y-axis. The effective friction angle can be calculated by using the inverse tangent of the failure (shear) stress divided by the normal stress. The “line” defining the effective shear strength of the soil will typically pass through the origin (i.e., 0, 0), which indicates no cohesion (i.e., $c=0$) in the sample. This is typical for most soils. Only cemented sands and some overconsolidated clays appear to have true effective cohesion (c') (Sabatini, 2002). If cohesion is indicated by the test results, and the samples are not cemented sands or overconsolidated clays, the samples were likely sheared too quickly to allow drainage of pore water during the test. Cohesion indicated by the test is often ignored, possibly conservatively, when performing slope stability analyses or foundation design. If cohesion is indicated by the direct shear test results but does not appear to be appropriate for the soil type, consideration should be given to performing additional direct shear tests.

Normal (vertical) stresses used for the test must be selected by the engineer and supplied to laboratory. Values of 4 ksf, 8 ksf, and 12 ksf are commonly used and are generally acceptable for most project conditions. The normal stresses used for the test should be selected to bracket the range of actual stresses anticipated at the project site. For example, assume a bridge pier founded on a spread footing bearing on soil is being considered during design, and direct shear testing is performed to estimate the soil shear strength in order to calculate bearing resistance. The estimated stress from the footing on the foundation soil is approximately 6 ksf; therefore, the commonly used values of 4, 8 and 12 ksf would be appropriate to use in the test for this situation. However, assume an embankment 100 feet high is proposed, and direct shear testing is being performed to estimate the shear strength of the foundation soil for slope stability analysis. Assuming an embankment unit weight of 120 pcf, the estimated stress on the foundation soil is 12 ksf (i.e., 100 feet x 120 pcf). Since the estimated stress in the field is 12 ksf, one of the normal stresses used in the laboratory should exceed 12 ksf. Possible normal stresses of 6 ksf, 10 ksf, and 14 ksf could be considered for this situation. Note that the effective overburden stress at the depth of the soil stratum being tested also needs to be added to the proposed embankment load when selecting laboratory normal stresses.

Direct shear testing can be performed on all soil types (granular, fine-grained, cohesionless and cohesive), but there are limitations on maximum soil particle size that can be used. The maximum permissible particle size is based on the size of the shear box that is used. Per the AASHTO test standard:

- The minimum width/diameter (square/circular box) of the shear box is 2.0-inches.
- The minimum thickness is 0.5-inch.
- The minimum diameter/width to thickness ratio is 2:1.
- The maximum particle size in the soil sample must be less than Six times the thickness of the shear box.

As an example, for a shear box with the minimum thickness of 0.5 inch, the maximum soil particle size that can be used in the test is 0.5-inch divided by 6, or 0.083-inch (2.1 mm), which is approximately equivalent to the No. 10 sieve opening.

The Department's Geotechnical Material and Testing Laboratory has the capability to perform direct shear testing. The Department's direct shear equipment includes a shear box that is 2.5 inches in diameter and 1-inch high. This size shear box is fairly typical of most labs. These dimensions meet the AASHTO requirements for minimum width/diameter, thickness, and diameter/width to thickness ratio. Based on AASHTO criteria, a soil sample with a maximum particle size of 0.17-inch (4.3 mm or approximately No. 4 sieve) can be used in this shear box. Shear boxes 12-inches wide by 6-inches thick and even larger exist, but they are not commonly found.

Disturbed soil samples used to remold samples for direct shear testing may contain particles that are larger (i.e., oversize) than that permitted by the AASHTO standard. These oversize particles must be removed prior to remolding the test samples. A few approaches can be taken for this situation. One approach is to simply remove/scalp all the oversize particles from the sample and test only the remaining portion of the sample. This approach will most likely yield a conservative/low shear strength value since the oversize particle sizes, which typically improve shear strength, were not used in the test. In this case consideration can be given to using a higher design shear strength value than was obtained from the laboratory test.

The other approach to deal with oversize particles is to replace the oversize particles with an equivalent amount (by weight) of coarse material that meets the AASHTO criteria. For example, if the maximum particle size permitted by the AASHTO standard is 4.3 mm, particles larger than this are removed from the sample, and an equivalent weight of particles that meet the AASHTO standards (say No. 4 to No. 10 material) is added to the sample. This approach is not provided in the AASHTO standard, but the US Army Corps of Engineers provides direction for this approach in "Laboratory Soils Testing", EM 1110-2-1906, dated November 30, 1970. This approach is included in [Section 4.8.4.16](#). Note that triaxial shear test samples are typically much larger than direct shear test samples, and therefore, are permitted to have larger particle sizes.

****NOTE TO REVIEWERS OF THIS CT: A PTM FOR DIRECT SHEAR IS CURRENTLY UNDER DEVELOPMENT. THIS PTM WILL INCLUDE, BUT IS NOT LIMITED TO, COARSE PARTICLE SUBSTITUTION, SAMPLE PREPARATION AND COMPACTION, AND SHEARING RATES. ONCE RESOLVED AND IMPLEMENTED, THIS SECTION WILL BE MODIFIED TO INCLUDE THIS PTM IN A FUTURE CT****

Undisturbed and remolded soil samples can be used for direct shear testing, but it is important to use the appropriate type of sample for the conditions being analyzed. Undisturbed soil samples should be used when direct shear testing is performed to estimate the in-situ strength of soil. In-situ soil strength is needed to:

- Analyze slope stability of an embankment foundation or a soil cut.
- Estimate bearing resistance for a spread footing supported on soil.
- Estimate the frictional resistance of a deep foundation.
- Estimate lateral earth pressure on retaining structures.

In-situ soils often times have characteristics or properties that cannot be replicated with remolded samples. For example, in-situ soils may be cemented or have residual/relic rock structure that provides strength to the soil mass. Conversely, the in-situ soil may contain thin, weak varves that decrease the soil mass strength. If a remolded sample is used to represent the in-situ soil, the strength indicated by the test result may be lower or higher than the in-situ strength, and sometimes significantly. However, when in-situ soils are cohesionless and undisturbed samples cannot be obtained, the use of remolded samples for direct shear testing for the above conditions will most likely be appropriate.

In addition to using remolded samples when Shelby tubes are not able to obtain a sample, remolded samples should also be used when the shear strength of proposed fill materials is needed. For example, assume a roadway project consists of cuts and fills/embankments, and the material from the cuts will be used construct the fills/embankments. If slope stability analyses of the proposed embankments are needed, then remolded samples should be used to estimate the shear strength of the embankment material. Similarly, in order to estimate the shear strength of a soil that will be used to construct a geosynthetic reinforced slope, the use of remolded samples is appropriate to perform the direct shear tests. Remolded samples are appropriate for these and similar conditions since they represent/model proposed field conditions. Remolded samples should be compacted to the same requirements anticipated for construction. For example, if the construction specifications require the soil embankment to be compacted to 95 percent of the maximum dry weight density and to within 2 percent of the optimum water content, then the remolded sample for testing should be compacted to these same specifications.

4.9.5 Residual Direct Shear

This test is performed to determine the residual shear strength of a soil. Residual shear strength is used in situations where there has been sufficient strain (e.g. slope failure) such that the material in question is past peak shear strength values. Residual shear strength is generally associated with fine-grained plastic soils. Residual and peak strengths are usually the same or very close for granular cohesionless materials. Residual shear strength values are generally not

used for the design of newly constructed earth structures, but rather for analysis of failed slopes or when mitigation of a failed slope involves adding resistance without reworking (removing and recompacting) the failed material. An example where residual shear strength parameters would be used for design would be when a wall is constructed in front of or intercepting the failure surface.

Neither AASHTO nor ASTM has a test method for residual direct shear. The United States Army Corps of Engineers EM 1110-2-1906, Appendix IX A, has a method that is often cited or referenced. This residual direct shear method is performed by repeatedly reversing the direction of shear until the residual (i.e., minimum) shear strength is obtained. Similar to the peak direct shear test, the residual direct shear test is performed slowly to ensure that the soil sample is consolidated and drained (i.e., no excess pore water pressure) during the test. This test is intended to represent or model the strength along a weak plane within the soil mass. This weak plane could be the result of past movement (e.g., sliding failure) or deposition (e.g., varves).

The United States Army Corps of Engineers EM 1110-2-1906, Appendix IX A method is not approved for use by the Department. There is concern that the method does not accurately model field conditions (i.e., continuous shear with large deformation in one direction). By repeatedly reversing the shear direction, there will be a realignment of particles with each direction reversal. As a result, orientation of particles parallel to the direction of shear may not be achieved as with large strain shear.

To determine residual shear strength parameters, the Department uses AASHTO T 236 “Standard Method of Test for Direct Shear Test of Soils under Consolidated Drained Conditions”. Residual values are obtained from the residual strength portion of the stress-strain plot of the direct shear test. It is necessary to run the direct shear test with sufficient displacement to assure residual shear has been observed (no change in shearing stress with continued displacement after peak shear stress been observed).

In situations where there is insufficient travel of the shear box to fully develop residual shear strength when using the standard AASHTO T 236 test method, the sample can be reversed back to the original position and then re-sheared. Realignment of particles in the reversal may result in some recovery of sample shear strength; however, it should be limited and should also be overcome with minimal strain. Additional shearing in the same direction (as the original shearing of the sample) will provide residual shear strength values. It is important that the sample reversal and re-shearing be conducted at the same strain rate as the original shearing to determine peak shear strength values.

4.9.6 Triaxial Shear, AASHTO T 296 and AASHTO T 297

The triaxial shear strength test can be used to estimate the strength of soil for various loading conditions, including:

- Unconsolidated-undrained
- Consolidated-undrained

- Consolidated-drained

Triaxial tests are primarily for fine-grained, cohesive soil with relatively low permeability. Coarse-grained soils and relatively high permeability soils that drain rapidly when load is applied are not appropriate for triaxial testing. Cohesive, fine-grained samples can contain coarse-grained particles; however, there are limitations on maximum particle size. Undisturbed and remolded samples can be used for this test.

The maximum acceptable particle size for use in the triaxial shear test is determined by the size of the triaxial cell used for the test. Per the AASHTO standard, the maximum acceptable particle size must be smaller than one sixth of the test sample diameter. Additionally, the test sample must be cylindrical, have a minimum diameter of 1.3 inches (33 mm) and have a height to diameter ratio between 2 and 2.5. Typical triaxial equipment found in laboratories can test samples up to 2.8 inches in diameter, which is the sample diameter obtained from a standard Shelby tube. Per the AASHTO standard, the minimum sample height for this diameter is 5.6 inches. Additionally, for this diameter, a maximum particle size of 0.5-inch (i.e., 2.8 inches divided by 6) can be used. This 0.5-inch (12-mm) particle is almost three times larger than the particle size permitted in a typical 1-inch high direct shear box. As discussed in [Section 4.9.4](#), there are guidelines for testing samples that have particle sizes larger than those permitted by the AASHTO test standard.

4.9.6.1 Triaxial Shear (UU), AASHTO T 296

Triaxial shear testing is used to estimate the shear strength of soil for the unconsolidated-undrained (UU) load condition. This strength is often referred to as the Q (i.e., quick) strength. This test provides an estimation of the undrained shear strength or cohesion. The UU test represents the condition in the field where load is applied to foundation soil (e.g., embankment, spread footing, etc.) and the soil does not have adequate time to drain excess pore water pressure and consolidate. Consequently, the load is carried by the water trapped within the voids/pores of the soil mass. UU test results can be used directly in slope stability analysis. Note that similar to the unconfined compression test, there are inherent problems with the UU triaxial test, so this test should not be relied upon solely for estimation of undrained shear strength (Sabatini, 2002).

The UU test is performed by applying a lateral confining stress (chamber pressure) and then applying an axial load without allowing the sample to drain. Normally three samples are sheared, and the chamber pressure is increased for each sample. The chamber pressures must be specified by the engineer and communicated to the laboratory, and these pressures should “bracket” the soil pressure expected in the field. Common pressures used in laboratories are 10, 20, and 40 psi (i.e., 1,440, 2,880 and 5,760 psf), but these must be checked for specific project needs. A more detailed discussion on selecting pressures for testing is included in [Section 4.9.4](#).

UU test results can be plotted using Mohr-Coulomb failure envelopes. Normal stress is plotted on the x-axis, and shear stress is represented on the y-axis. The ideal test results will yield a single line that is tangent to all of the Mohr-Coulomb envelopes and parallel to the x-axis. The shear strength is equal to the y-intercept (i.e., cohesion), and an internal friction angle of zero is represented by the line parallel to the x-axis (i.e., no slope). Due to possible variations

between samples degree of saturation and void ratio, samples may not exhibit constant undrained shear strength at various normal/axial loads. Where this variation is significant, each test should be evaluated individually, and undrained shear strength should be determined for each sample by assuming an internal angle of friction of zero. The range of undrained shear strengths will have to be analyzed, and an appropriate value selected for design. Where the variation is minimal, a best fit line taking all results into account should be appropriate.

As previously indicated UU triaxial testing can be performed on either undisturbed or remolded samples, but it is important to use the appropriate type of sample for the condition being analyzed. UU testing is normally performed to estimate the shear strength of in-situ soil that will be loaded by an embankment or a footing. Therefore, undisturbed samples should be used for this type of UU testing. Since UU testing is not generally performed to estimate the shear strength of compacted fills/embankments, remolded samples are typically not appropriate. However, if undisturbed samples are not available, the use of remolded samples can be considered for testing.

4.9.6.2 Triaxial Shear (CU), AASHTO T 297

Triaxial shear testing is used to estimate the shear strength of soil for the consolidated-undrained (CU) load condition. This strength is often referred to as the R (i.e., rapid) strength. This test provides an estimation of total shear strength. The CU test represents the condition in the field where load is applied to foundation soil (e.g., from an embankment, spread footing, deep foundation, etc.) and the foundation soils have had time to consolidate but excess pore water pressure is still present. This strength condition is also applicable where foundation soils have been consolidated prior to construction loading, such as from glaciation. The CU strength phase occurs between the UU and the CD (consolidated-drained) strength phases. CU test results can be used directly in slope stability analysis and foundation design.

The CU test is performed similarly to the UU test, but the CU test is performed by consolidating the sample before shearing. Once consolidated, a lateral confining stress (consolidation pressure) is applied, and then an axial load is applied to shear the sample. Similar to the UU test, the sample is not permitted to drain during testing. Normally three samples are sheared, and the consolidation pressure is increased for each sample. The consolidation pressure, which is the difference between the chamber pressure and the back pressure, must be specified by the engineer and communicated to the laboratory. These pressures should “bracket” the soil pressure expected in the field. Common consolidation pressures used in laboratories are 10, 20, and 40 psi (i.e., 1,440, 2,880 and 5,760 psf), but these must be checked for specific project needs. A more detailed discussion on selecting pressures for testing is included in [Section 4.9.4](#).

CU test results can be plotted using Mohr-Coulomb failure envelopes. Normal stress is plotted on the x-axis, and shear stress is represented on the y-axis. The ideal test results will yield a single line that is tangent to all of the Mohr-Coulomb envelopes, which is similar to the UU test results. The cohesion portion of the soil shear strength is equal to the value where the line crosses the y-axis. Unlike the UU test results, the CU test results will typically yield a positive-sloped line, indicating that the soil also has a frictional component to its shear strength. Due to possible variations between degree of saturation and void ratio, samples may not exhibit a

uniform total shear strength (i.e., single line cannot be drawn tangent to all sample Mohr-Coulomb failure envelopes). Where the variation is minimal, a best fit line taking all results into account should be appropriate. Where the variation is significant, the test results should be reviewed, and additional testing should be performed if necessary.

CU triaxial testing can be performed on either undisturbed or remolded samples, but it is important to use the appropriate type of sample for the condition being analyzed. CU testing is normally performed to estimate the shear strength of in-situ soil that will be loaded by an embankment or a footing. Therefore, undisturbed samples should be used for this type of CU testing. Since CU testing is not generally performed to estimate the shear strength of compacted fills/embankments, remolded samples are typically not appropriate. However, if undisturbed samples are not available, the use of remolded samples can be considered for testing.

4.9.6.3 Triaxial Shear (CU with pore pressure measurement), AASHTO T 297

Triaxial shear testing is used to estimate the shear strength of soil for the consolidated-drained (CD) load condition. This strength is often referred to as the S (i.e., slow) strength. This test provides an estimation of effective shear strength. The CD test represents the condition in the field where load is applied to foundation soil (e.g., from an embankment, spread footing, deep foundation, etc.) and the foundation soils have had time to consolidate and drain (i.e., no excess pore water pressure). The CD strength phase occurs after the CU strength phases. CD test results can be used directly in slope stability analysis and foundation design. Due to the significant length of time required to perform the CD test, a CU test with pore pressure measurement can be performed as an alternative. This test method is much quicker than the CD test and provides not only an estimate of the effective shear stress, but the total shear strength as well.

The CU with pore pressure measurement test is performed the same as the CU test except pore water pressure is measured during the test. This pore pressure measurement allows the estimation of the effective (i.e., drained) shear strength. Although typically more expensive than the “plain” CU test, the CU with pore pressure measurement test is generally preferred since both total and effective shear strength are determined when pore pressure is measured. Note that the CD test can be performed by shearing the sample extremely slowly to allow consolidation and pore pressure dissipation; however, this method is typically not used because it is too time consuming and costly. Additionally, this method only provides effective shear strength and not total shear strength.

CU with pore pressure measurement test results can also be plotted using Mohr-Coulomb failure envelopes. Total shear strength results are plotted the same as discussed in the CU Triaxial Shear discussion above. Effective shear strength results from the test are plotted similarly. These results should yield a shear strength with less, and possibly zero, cohesion, but a higher internal friction angle compared to the total shear strength. Similar to the “plain” CU test, due to possible variations between samples degree of saturation and void ratio, samples may not exhibit a uniform total and/or effective shear strength. Where the variation is minimal, a best fit line taking all results into account should be appropriate. Where the variation is significant, the test results should be reviewed, and additional testing should be performed if necessary.

CU triaxial testing with pore pressure measurement can be performed on either undisturbed or remolded samples, but it is important to use the appropriate type of sample for the condition being analyzed. CU with pore pressure measurement testing is normally performed to estimate the shear strength of in-situ soil that will be loaded by an embankment or a foundation. Therefore, undisturbed samples should normally be used for this testing. Since CU with pore pressure measurement testing is not generally performed to estimate the shear strength of compacted fills/embankments, remolded samples are typically not appropriate. However, if undisturbed samples are not available, the use of remolded samples can be considered for testing.

4.9.7 Comparison of Direct Shear and Triaxial Shear Testing

It is often desirable and beneficial to perform shear strength testing using both the direct shear and triaxial shear test methods, but some situations/projects may not warrant both. As previously discussed direct and triaxial shear tests can be performed on both undisturbed samples and remolded samples. The direct shear test can be performed on both granular, although limited particle size, and fine-grained (cohesive and cohesionless) materials. The triaxial shear test can only be performed on soils with sufficient cohesion so samples maintain their shape while unconfined prior to placing in the triaxial cell. Triaxial test samples can contain granular material, and typically can have larger particle sizes compared to direct shear samples.

The direct shear test is a drained (i.e., no excess pore pressure) test, so only drained or effective shear strength can be obtained from this test. The direct shear test cannot be used to estimate undrained or total shear strength. Conversely, the triaxial shear test method can be used to estimate undrained, total, and effective shear strength.

One of the main differences between the direct and triaxial shear test is the orientation of the failure plane produced by the test. The direct shear test forces the soil sample to shear along a horizontal plane (i.e., normal/perpendicular to the axial load) between the two halves of the shear box, whereas the triaxial shear test forces the sample to shear on an inclined plane (i.e., at an angle to the axial load). This difference in failure plane orientation must be considered when naturally occurring weak horizontal planes from varves, ancient failure planes, etc. are present in the soil stratum being tested. The direct shear test can be used to estimate the shear strength along these weak planes, assuming samples with these planes can be obtained. The triaxial shear test would most likely not detect the weakened strength since the failure plane would cut across, and not along, the naturally occurring weakened plane. However, even where weakened planes are present in the soil, it will often be necessary/beneficial to use the triaxial test method to obtain the “higher end” range of the shear strength for potential failure planes that cut across existing weakened planes.

Some advantages of performing the direct shear test in lieu of the triaxial shear test include:

- Estimates both peak and residual shear strength
- Typically, less expensive and quicker to perform
- Less soil sample is needed

- Shear strength along existing weakened planes can be estimated
- More common/more laboratories are capable of performing.

Some advantages of performing the triaxial shear test in lieu of the direct shear test include:

- Larger test samples are possibly more representative of in-situ conditions
- Undrained, total and effective shear strength can be estimated
- Typically, can accommodate larger particle sizes.

4.9.8 Rock Unconfined Compressive Strength

Rock core samples chosen for testing should have a length-to-diameter ratio (L/D) of 2.0 to 2.5. When samples of rock core with the required L/D are not available, samples with an L/D as close to 2.0 may be used and the apparent compressive strength corrected in accordance with the following equation from a previous ASTM standard (ASTM D2938-86):

$$C = C_a / \left(0.88 + \left(\frac{0.24b}{h} \right) \right)$$

where:

C = computed compressive strength of an equivalent L/D = 2 specimen,

C_a = measured compressive strength of the specimen tested,

b = test core diameter

h = test core height

The use of this correction should be clearly documented as it is not included in the current ASTM.

4.9.9 Testing and Monitoring Domestic Water Supplies

Testing and monitoring of domestic water supplies, which include wells and springs, must be considered on projects where blasting will be conducted for rock excavation in the vicinity of domestic water supplies. Although less commonly done, testing and monitoring should also be considered when other construction activities, including earthwork and pile driving, are planned in the vicinity of domestic water supplies. If a public water supply, such as a well owned by a city, municipality, or private company, is located in the vicinity of a project site, more stringent testing and monitoring may be required. When a public water supply is involved, close coordination with the well owner and PADEP is required to develop an appropriate and amicable testing and monitoring program.

The effects of blasting and other construction activities on water supplies is dependent upon numerous factors, including: type of construction activity, distance and location of water supply from construction activity, type of water supply (e.g., shallow spring or well, deep well, etc.), well construction (e.g., uncased or shallow cased well, deep cased well, etc.), geology, and regional groundwater flow. These and other factors must be considered when determining which water supplies should be tested and monitored. Typically, water supplies located within approximately 1,000 feet of blasting should be tested and monitored, although some projects

may warrant testing water supplies more than 1,000 feet from blasting activities. For construction activities other than blasting, a distance of less than 1,000 feet is likely reasonable, but as previously discussed, this must be determined on a project by project basis.

Water supplies should be tested prior to construction activities to establish baseline information that can be compared to information obtained during and/or after construction. In some cases, testing may be beneficial more than once prior to construction. For example, testing could be performed once during wetter seasons and once during dryer seasons. Water supplies should also be tested shortly after construction activities are complete (i.e., within 8 weeks) in the area of the water supply, and tested as needed during construction to address concerns raised with specific water supplies. This testing can be specified in the construction contract documents (i.e., performed by the contractor) or it can be performed by a consultant retained directly by the Department. In some cases, the baseline testing (i.e., testing completed before construction) may have to be performed by a consultant, and any subsequent testing be performed by the contractor.

The following guidelines are provided to develop the water supply testing and monitoring program:

- The overall intent of the testing and monitoring program is to obtain available water supply information and estimate the quality of water from the water supply.
- Interview the property owner to obtain information about the water supply, including: year constructed, well depth and diameter, casing depth, pump depth, the well yield that was estimated by the driller during construction of well, and any known well issues/problems (e.g., turbidity, yield, contaminants, etc.). The underside of the well cap, if accessible, may contain well information. Note that it is not recommended to measure depth of water inside the well casing due to concerns with possible contamination.
- Collect water samples using sample collection and preservation techniques specified by the testing laboratory.
- Collect water samples prior to treatment system (e.g., softener, disinfection system, etc.)
- Allow water to run approximately 10 to 15 minutes before collecting samples to ensure a representative sample from the water supply is obtained.
- At a minimum perform tests for the following: pH, temperature, specific conductance, turbidity, total coliforms, fecal coliforms/e. coli, nitrate, nitrite, chloride, hardness, iron, and sulfates. pH, temperature, specific conductance and turbidity should be measured in the field, and the other tests are performed in the laboratory. Tests for additional parameters may be done based on site specific project conditions.
- Use a PADEP certified laboratory to perform tests.
- Executing the water supply testing and monitoring program requires coordination with the property owner to accommodate their schedule. The interview with the property owner can either be conducted over the phone or in person.

The consultant scope of work must clearly indicate the anticipated work associated with the testing and monitoring of water supplies, including: number of water supplies to be tested/monitored, number of times each water supply will be tested, specific tests to be performed, and deliverable(s). The deliverable(s) must include the field and laboratory test results. It is also recommended that the deliverable include a brief report/narrative that summarizes the test results, identifies areas of concerns or potential impacts to water supplies from planned construction, and provides recommendations for modifications to the original scope of work (e.g., additional round/phase of testing, testing for additional parameters, etc.). Typically a deliverable is provided after each phase of testing.

Note that the preceding discussion is focused on testing and monitoring for construction activities. Some projects may warrant some level of water supply study during the preliminary or final design phase. For instance, if multiple alignments are being considered, it may be necessary to investigate the number of water supplies that could potentially be impacted by the various alignments. Similarly, the number of water supplies that could be impacted by a project may need to be investigated to estimate the cost of repairing or replacing the impacted supplies. In these instances, identifying the water supplies and interviewing the owner may be sufficient. Water sampling and testing may not be needed.

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CHAPTER 5 – SOIL AND ROCK PARAMETER SELECTION

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5.1 INTRODUCTION

This chapter of the publication provides guidelines, recommendations, and considerations for selecting soil and rock parameters using the results from the subsurface exploration and laboratory testing programs. The soil and rock parameters selected for use in geotechnical analyses (e.g., bearing resistance, settlement, slope stability, etc.) are fundamental to the final geotechnical recommendations. Therefore, it is imperative that sufficient and good quality subsurface information be obtained so that these parameters can be estimated as accurately as possible. **Geotechnical engineering analyses are meaningless if the soil and/or rock parameters used in the analyses are not representative of the project subsurface conditions. Additionally, resistance/safety factors prescribed in design manuals are based on the assumption that soil/rock parameters have been estimated from good quality data.**

5.1.1 Purpose

The purpose of this chapter is to identify appropriate methods for assessing soil and rock properties and describing how to use soil and rock property data to establish the final soil and rock parameters for use in geotechnical analyses. Soil parameters needed for geotechnical design on roadway and bridge projects include unit weight, soil strength, soil deformation (i.e., settlement), hydraulic conductivity (i.e., permeability), and many others. [Table 5.1.1-1](#) provides information on soil and rock parameters typically used on roadway and bridge projects.

Table 5.1.1-1 – Summary of Soil and Rock Parameters

Parameter	Symbol	Unit	Applicable Material	Correlation Estimate	Direct Estimate ¹	Geotechnical Analyses
Angle of Internal Friction of Drained Soil	ϕ (phi)	degree	Cohesionless Soil – Granular including and Non-Plastic Silt	Classification, SPT, CPT	Direct Shear, Triaxial Shear ²	Bearing Resistance, Friction Pile/Drilled Shaft in Soil Resistance, Slope Stability, Sliding Resistance, Lateral Earth Pressure
Drained (Effective) Shear Strength of Cohesive Soils	ϕ'	degree	Cohesive Soil – Clay and Plastic Silt	None		
	c'	kcf				
Undrained (Total) Shear Strength of Cohesive Soils ³	c, s_u	kcf	Cohesive Soil – Clay and Plastic Silt	None	Triaxial Shear, (CU or UU) Field Torvane, Pocket Penetrometer	
Unit Weight	γ	kcf	All Soils	Classification, SPT, CPT	Unit Weight Test ⁴	

Parameter	Symbol	Unit	Applicable Material	Correlation Estimate	Direct Estimate ¹	Geotechnical Analyses
Compression Index	C_c	dim	Cohesive Soil – Clay and Plastic Silt	None	Consolidation Test	Consolidation (long term) Primary Settlement
Recompression Index	C_r	dim				
Void Ratio	e_o	dim				
Coefficient of Consolidation	c_v	ft ² /year				
Secondary Compression Index	C_α	dim	Organic Cohesive Soil – Clay and Plastic Silt	None	Consolidation Test	Consolidation (long term) Secondary Settlement
Young’s Modulus (Elastic Modulus)	E_s	ksi	All Soils	Classification, SPT, CPT, Field Torvane, Pocket Penetrometer	PMT, DMT	Elastic (immediate) Settlement
Poisson’s Ratio	ν	dim				
Permeability (Hydraulic Conductivity)	k	Length/time (cm/sec)				
				Classification Gradation	Permeability Test, Field Infiltration Test	Dewatering Design, Drainage/Filter design, Stormwater Design
Rock Unconfined Compressive Strength	q_u	Stress (ksf, psi, etc.)	Rock	Point Load Test, Field Description (rock type and condition)	Unconfined Compressive Strength Test	Foundation Design, Excavation (rippability, blasting)
Rock Mass Shear Strength	τ	ksf	Rock	GSI ⁵ , Hoek Brown Failure Criterion	None	Passive Resistance of Soldier Piles
Rock Mass Modulus	E_m	ksi	Rock	GSI ⁵	None	Settlement
Grout to Ground Bond Strength	α_b	ksf	Rock	Rock Description	Pile Load Test	Micropile Grout-to-Ground Bond Resistance

- Notes: 1. Field and laboratory tests are described in Chapters 3 and 4, respectively
2. CU with pore pressure measurements
3. Cohesion (c) = Undrained Shear Strength (s_u)/2
4. Cohesive soil at in-situ moisture content
5. Geologic Strength Index for Jointed Rocks

5.1.2 Approach to Selecting Parameters

After the subsurface exploration and laboratory testing programs are complete, soil and/or rock parameters need to be selected in order to perform the necessary geotechnical design

calculations/analyses. Selecting these parameters must be done using a logical, systematic approach and must be documented in the appropriate geotechnical design report, as discussed in [Chapter 1](#) of this publication. This documentation will greatly aid reviewers of the report.

The following general approach should be used to select geotechnical design parameters:

1. Determine the geotechnical soil and rock parameters that are needed
2. Finalize/Update the subsurface profile
3. Select the soil and rock parameters and perform analyses/design

5.1.2.1 Determining the Necessary Geotechnical Soil and Rock Parameters

All projects are unique, and therefore, the geotechnical design parameters needed for design will vary from project to project. The required geotechnical design parameters were already considered during planning of the subsurface exploration and laboratory testing programs. However, these parameters must be reconsidered based on the information obtained from both programs because the parameters required for design may differ based upon the actual subsurface conditions. For example, perhaps consolidation settlement was not anticipated on a project; however, during the subsurface exploration, saturated clay was encountered and undisturbed (Shelby tube) samples were obtained to assess the potential for consolidation settlement in the clay material. Classification and consolidation test results showed that consolidation settlement would be a concern, and parameters relative to quantifying the consolidation settlement would be required for design.

5.1.2.2 Finalizing/Updating the Subsurface Profile

Final Geotechnical Engineering Reports and final roadway plans will include subsurface information graphically presented on the roadway profile in accordance with Design Manual, Part 3 (DM-3), Chapter 5. Structure Foundation Geotechnical Reports are to include subsurface profiles in the appendices in accordance with [Chapter 1, Section 1.5.7](#) of this publication. Working subsurface profiles (and cross sections as appropriate), prepared early in the design process, and updated as additional information becomes available are an invaluable tool in geotechnical design.

The working subsurface profile(s) for the structure(s)/roadway(s) will help select soil and/or rock samples for laboratory testing. Showing the laboratory test results on the working subsurface profile, provides a useful summary of data, and can help to distinguish strata based on similarities/differences of laboratory test results.

Note that as discussed in the [Chapter 4](#) of this publication, the laboratory test results must be reviewed by the Project Geotechnical Manager (PGM) as soon as the test results become available. Test results that are suspect, do not agree with the results from testing of similar materials, and/or vary from anticipated results must be carefully reviewed. Based on the outcome of the review, the PGM must determine if the test results are adequate and of good quality for the selection of geotechnical design parameters, or if additional testing is required. Any additional testing performed must be first approved by the District Geotechnical Engineer (DGE).

In addition to subsurface information, proposed bottom of structure footing elevations, approximate pile/drilled shaft tip elevations, proposed roadway grade, etc., will help determine the parameters needed for the various subsurface strata to perform the geotechnical analyses. Parameters may not have to be determined for all the strata; parameters are determined relative to the analysis performed and affected soils and rock. For example, competent strata beneath the zone of influence from a given load usually will not require the development of geotechnical parameters. However, near surface soils that will be stripped prior to construction or that are not subject to a structure load will most likely not influence geotechnical analyses for structure foundation design, but parameters would be needed to evaluate slope stability or excavation support during construction.

5.1.2.3 Selecting Soil and Rock Parameters and Performing Analyses/Design

Once the field and laboratory test results/data are reviewed and summarized on the subsurface profile(s), the soil and rock parameters can be selected. Geotechnical parameters of soil and rock can be determined in several ways, including: using in-situ field testing during subsurface exploration (e.g., vane shear or pressure meter tests, etc.), using laboratory performance testing (e.g., shear strength testing, consolidation testing, etc.), correlations using field tests and laboratory classifications/index properties (e.g., SPT data and soil classification, etc.) in conjunction with local/past experience with similar materials (e.g., glacial deposits, red beds, etc.), or back analysis. Back analysis is often used with slope failures, embankment settlement, or excessive settlement of existing structures. Often the determination of these properties is aided by correlations with index tests or experience on other projects. Back analysis can be used early in the design process to estimate soil properties (before laboratory testing results are available), for evaluating the viability of various remediation options, and to verify the selected parameters for final design. Slope stability analysis and selecting and assessing slope parameters will be addressed in more detail in [Chapter 6](#) of this publication.

Geotechnical analyses/design can proceed once parameters are selected. If the results of the analyses/design do not meet expectations or do not appear reasonable, revisit the rationale used to select the parameter(s) and the types of tests performed to obtain the parameters. Do not adjust parameters simply to match expectations. In cases where data are scattered and parameter(s) could have significant variation, consider doing analyses with both high and low values to better estimate the influence of the parameter on the analysis/design. Reference [Section 5.6](#) of this publication for an in-depth discussion on the selection of final design values.

5.2 UNDERSTANDING SOIL AND ROCK CONDITIONS

The engineering behavior of soil and rock is its general response to mechanical, hydraulic, or other types of loading. The engineering response of undisturbed and compacted soils and in-situ rock are addressed below. Undisturbed soils and in-situ rock respond to loading only as a result of geologic processes or environmental factors unrelated to projected construction activities. Compacted soils derive their engineering behavior in response to intervention by construction activities. In short, factors such as a sites' stress history, especially relative to soils, can have a major impact on material properties and selected design parameters.

5.2.1 Regional Soil Conditions

Like in-situ rock, undisturbed soils derive their engineering behavior mostly as a result of the geologic environment controlling their development. Within Pennsylvania, four primary and several secondary geologic processes are responsible for the development of the surficial soil cover. The primary geologic processes include:

1. In-place weathering of rock units (residual soils)
2. Deposition of soil by water (alluvial soils)
3. Deposition of soil by glacial processes (glacial soils)
4. Deposition of soil by gravity (colluvial soils)

Residual soils are the result of the in-place weathering (i.e., breakdown) of bedrock. Since the majority of Pennsylvania is underlain by sedimentary rock, the primary source of residual soils in Pennsylvania is the weathering of non-carbonate and carbonate sedimentary rocks. Alluvial soils are transported by running water. These soils have been deposited within and adjacent to stream and river systems throughout Pennsylvania. Glacial soils are predominate in the northeastern and northwestern sections of Pennsylvania. These soils were deposited by advancing and retreating glaciers. Glacial soils include: glacial till (i.e., unsorted mixture of soil transported and deposited by glacial ice), glacial outwash (i.e., typically stratified gravel, sand and silt deposited by melting glacier ice), glacial fluvial deposits (i.e., material deposited by streams flowing from melting glacier ice), and glacial lacustrine deposits (i.e., clay, silt and sand, often laminated, deposited by melting glacier ice into glacial lakes). Colluvial soils are transported predominantly by gravity, and also by surface water runoff. These soils include boulder to clay size material, and are found on and at the base of slopes.

General engineering characteristics of sedimentary and transported soils are presented in [Table 5.2.1-1](#). This table classifies and defines the soil geological processes found in Pennsylvania and provides brief pertinent engineering characteristics that may aid in the understanding and selection of soil properties.

Table 5.2.1-1 – General Engineering Characteristics of Principal Soil Deposits

Geologic Process	Principal Soil Deposits	Pertinent Engineering Characteristics
<p>Residual Material formed by disintegration of underlying parent rock or partially indurated material.</p>	<p>Residual sands and fragments of gravel size formed by solution and leaching of cementing material, leaving the more resistant particles; commonly quartz.</p>	<p>Generally favorable foundation conditions</p>
	<p>Residual clays formed by decomposition of silicate rocks, disintegration of shales, and solution of carbonates in limestone. With few exceptions becomes more compact, rockier, and less weathered with increasing depth. At intermediate stage may reflect composition, structure, and stratification of parent rock.</p>	<p>Variable properties requiring detailed investigation. Deposits generally present favorable foundation conditions.</p>

Geologic Process	Principal Soil Deposits	Pertinent Engineering Characteristics
<p>Alluvial Material transported and deposited by running water.</p>	<p>Floodplain is a deposit laid down by a stream within that portion of the valley subject to inundation by flood waters.</p>	
	<p>Point bars are alternating deposits of arcuate (bow-like) ridges and swales (lows) formed on the inside or convex water bank of mitigating river bends. Ridge deposits consist primarily of silt and sand; swales are clay filled.</p>	<p>Generally favorable foundation conditions; however, detailed investigations are necessary to locate discontinuities. Flow slides may be a problem along riverbanks. Soils are quite pervious.</p>
	<p>Channel fill is a deposit laid down in abandoned meander loops isolated when rivers shorten their courses. Composed primarily of clay; however, silty and sandy soils are found at the upstream and downstream ends.</p>	<p>Fine-graded soils are usually compressible. Portions may be very heterogeneous. Silty soils generally present more favorable foundation conditions, but are moisture sensitive and tend to pump.</p>
<p>Alluvial (cont.)</p>	<p>Backswamp is the prolonged accumulation of floodwater sediments in flood basins bordering a river. Materials are generally clays but tend to become more silty near riverbank.</p>	<p>Relatively uniform in a horizontal direction. Clays are usually subjected to seasonal volume changes.</p>
<p>Glacial Material transported and debris, deposited by glaciers, or by meltwater from the glacier.</p>	<p>Glacial till is an accumulation of debris deposited beneath, at the side (lateral moraines), or at the lower limit of a glacier (thermal moraine). Material lowered to ground surface in an irregular sheet by a melting glacier is known as a ground moraine.</p>	<p>Consists of material of all sizes in various proportions from boulders and gravel to clay. Deposits are unstratified. Generally present favorable foundation conditions, but rapid changes in conditions are common.</p>
<p>Colluvial Material transported and deposited by gravity</p>	<p>Talus is deposits created by gradual accumulation of unsorted rock fragments and debris at base of cliffs.</p>	<p>Previous movement indicates possible future difficulties. Generally unstable foundation conditions.</p>

5.2.2 Special Problematic Soils and Rocks

Special problem soils and rocks of importance within Pennsylvania include micaceous soils, pyritic soils and rock, soils containing non-plastic silts, peats, slags, and slaking rock. These special problem materials must be considered when selecting soil parameters.

5.2.2.1 Micaceous Soils

Micaceous soils are derived from the weathering of many igneous and metamorphic rock formations. Micaceous soils are common in the eastern and southeastern parts of Pennsylvania. The presence of mica in undisturbed soils can result in low shear strength making the soils susceptible to accelerated erosion and landslides, and therefore, necessitate the use of relatively flat (i.e., flatter than 2H:1V) cut slopes. When micaceous soils are used to support foundations, higher than normal elastic settlements are possible due to the compressibility of mica. If

compacted fills are constructed using micaceous soils, particularly if micaceous silts are to be compacted, careful control of moisture content may be required to meet compaction criteria.

5.2.2.2 Pyritic Soils and Rocks

Pyritic soils are derived from the weathering of pyritic black shales commonly associated with coal-bearing strata. Within Pennsylvania, pyritic rocks and residual soils occur from weathered Pennsylvanian Age Pottsville, Allegheny, Monongahela and Conemaugh Groups, and the Permian Age Dunkard Group. Pyritic soils and rock may include material derived from nearby non-black and non-carbonaceous shale strata in the vicinity of coal horizons. Pyritic soils and rock are associated with problems related to acid runoff, swelling, or heaving due to the oxidization of sulfide minerals, and concrete deterioration due to low pH. These problems can be minimized or eliminated by over excavation and removal, chemical treatment, or the use of sulfate resistant cement. Further discussion of acid bearing rock is included in this publication in [Chapter 2, Section 2.3.3.8](#), and [Chapter 10](#).

The presence of pyritic material on a project may require coordination with Pennsylvania Department of Environmental Protection (DEP) and disposal and/or treatment may become very expensive.

5.2.2.3 Peats

Peat is a naturally occurring, highly organic material derived from plant materials. In Pennsylvania, peat deposits are associated with the glacial deposits (mostly in the northeast) and result from the accumulation of organic debris in shallow lakes or kettles within a pro glacial environment. Peats are very compressible. Embankments can be constructed over peat deposits if provisions are included to limit the rate of construction allowing settlement to occur incrementally, or if geotextiles are incorporated at the base of the fill strengthening the soil for support of the embankment. Structures must not be supported on peat.

5.2.2.4 Non-plastic Silts

If present in sufficient quantities, the presence of non-plastic silts can clog soil voids/pores and render the soil marginally draining. The effects on the soil from this condition may include frost susceptibility, increased seepage pressures, and liquefaction. Compacting soil containing significant amounts of non-plastic silts is difficult due to moisture sensitivity. Additionally, such material provides no cohesion. Compaction of non-plastic silt is best done with thin lifts.

5.2.2.5 Slag

Slags from steel production are used in some Department Engineering Districts as a substitute for soil backfill below pavements and retaining structures. Problems of expansion are associated with open-hearth slags, which must be avoided as they contain free calcium oxide (CaO) or magnesium oxide (MgO). Blast furnace slags are generally stable and can be used for construction purposes unless high quantities of calcium are present.

5.2.2.6 Slaking Rocks

Some sedimentary rocks common to Pennsylvania degrade quickly when exposed to the elements or disturbed. Rock types prone to slaking include claystones, mudstones, shales and some siltstones. Slaking behavior impacts the engineering performance of rock cut slopes, foundation subgrades, pile resistance and embankment rock fills. A further discussion of slaking rock is included in [Section 5.5.4.5](#).

5.2.3 Compacted Soils

Information regarding the engineering behavior of compacted soils is required when subgrade soils cannot support structures and fills or when engineered fills for embankments or backfills are needed. Compaction requirements are developed considering the engineering properties needed for a particular situation (e.g., strength, compressibility, and/or permeability). Laboratory compaction tests to establish moisture content-dry unit weight relationships are used in combination with field measurements to determine whether field compaction efforts meet the compaction control criteria. A detailed discussion on compaction of soils will be presented in [Chapter 9](#) of this publication.

Principal soil properties affected by compaction include settlement, shearing resistance, permeability, and volume change. Discussions of compacted soils are included in [Section 5.5](#) of this publication.

5.3 STANDARD PENETRATION TEST (SPT) AND N-VALUE

As discussed in [Chapter 3](#) of this publication, the most common method in Pennsylvania of obtaining disturbed soil samples is split-barrel sampling. Split-barrel sampling, in conjunction with Standard Penetration Testing (SPT), is an ideal method to: visually classify soil, estimate soil density/consistency and natural moisture content, and collect disturbed samples for laboratory testing. Regarding SPT data, N-values obtained in the field depend on the equipment used, the skill of the operator, and the soil type/gradation encountered. Correct N-values before they are used in design so that they are consistent with the design method and correlations being used. Many of the correlations developed to determine soil properties are based on N_{60} values. Guidance on the use of correction factors is provided in the following subsections.

5.3.1 Soil Type/Gradation

SPT can be performed in most soil deposits; however, SPT data (i.e., N-values) is particularly useful in cohesionless sands. This is because the most useful/reliable correlations between SPT data and engineering properties of soil are in cohesionless sands.

Caution should be used when using N-values obtained in gravelly soil because the presence of gravel can be problematic for split-barrel sampling and SPT. When gravel is encountered during sampling, and in particular medium- to coarse-grained gravel, the sampler will sometimes push the gravel as the sampler is being advanced instead of the gravel entering

the split-barrel. When the gravel is pushed, it increases the resistance to barrel penetration, and therefore, artificially inflates the blow counts and estimates of friction angles. Additionally, coarse gravel often prohibits the collection of a representative soil sample because the coarse gravel blocks the barrel opening and prevents smaller-grained soil from entering the barrel. Therefore, when sampling soils containing gravel, SPT N-values need to be closely scrutinized. If sample recovery is low, it could be an indication that gravel was pushed and/or blocked the barrel opening and therefore, resulting N-values may be artificially high. Also, if sporadic high N-values are recorded in a gravelly stratum, it could be an indication that the higher N-values are the result of gravel being pushed ahead of the barrel.

Although SPT is and should be performed in cohesive soil, SPT data are not useful for estimating the shear strength (i.e., cohesion or friction) of cohesive soil. SPT data are also generally not useful for estimating the shear strength of silt (plastic or non-plastic) due to the dynamic nature of the test and resulting rapid changes in pore pressures and disturbance within the deposit. Additionally, SPT data recorded in saturated silts and fine sands below the ground water level can be artificially low due to dynamic effects from the hammer energy (i.e., liquefaction).

5.3.2 Importance of Standardization for Standard Penetration Testing

There are numerous variables with respect to procedure and equipment used that affect the results of standard penetration testing, and therefore affect the SPT N-value. Some procedure variables that can affect SPT results include: preparation and diameter of the borehole, number of rope wraps on the cathead to lift the hammer, height hammer is dropped, and accuracy of counting and recording hammer blows. Equipment variables include: dimensions (i.e., diameter and length) of split-barrel sampler, condition of barrel and tip, split-barrel sampler equipped with operating check valve, condition and length of drill rods, condition of rope, hammer type (i.e., donut, safety, and automatic), and hammer condition. These and other variables determine the amount of energy that is delivered to the tip of the split-barrel sampler, which directly influences the N-value. The relationship between this energy and the N-value is inverse, in that for identical subsurface conditions, as the energy delivered to the split-barrel sampler tip increases, the N-value decreases. Due to these numerous variables and their direct effect on N-value, it is imperative that SPT testing be done in strict accordance with Publication 222, Section 202 so that representative N-values are obtained.

There are numerous reasons why it is critical that SPT always be performed using the standard procedures and equipment prescribed in ASTM D 1586 (and where superseded by Publication 222, Section 202). N-values obtained from borings on a project site are routinely used with published correlations between N-values and engineering properties to perform geotechnical design. The N-values reported in published correlations were obtained in accordance with similar procedures as prescribed in ASTM D 1586, unless otherwise noted. Therefore, in order to properly use these published correlations, N-values obtained from project borings must also be obtained with this “standard energy” (i.e., in accordance with ASTM D 1586). Another reason it is critical to use the “standard energy” to perform SPT is so N-values obtained throughout a project site can be used to help delineate various strata. If variable energies are used across a site to perform SPT, the N-values will not be useful to estimate

differences/similarities between strata. Furthermore, correlations between project N-values and site specific laboratory testing will not be representative if the “standard energy” is not used to perform SPT.

5.3.3 SPT Hammer Efficiency Correction

Assuming that SPT is performed with equipment and procedures that meet ASTM D 1586, and where superseded by Publication 222, Section 202, the variable that has the most impact with respect to the actual energy delivered to advance the split-barrel sampler is the type of hammer system used. One of three hammer systems is typically used to perform SPT, and they include: donut hammer, safety hammer and automatic hammer. Donut hammers are the oldest and least frequently used hammer system for performing SPT. Safety hammer systems, similar to donut hammers, are manually raised and lowered with a rope and cathead; however, these hammer systems include safety features that donut hammers do not have. Automatic hammers are raised and lowered mechanically; therefore, a rope and cathead are not needed. Automatic hammers are advantageous since they typically deliver more consistent energy when compared to the manually operated systems because variables associated with manually raising and lowering the hammer are eliminated. Also, these hammer systems have additional safety features compared to safety hammer systems

The energy delivered by a hammer system is measured in terms of efficiency. Hammer system efficiency is very important because most published correlations between SPT N-value and soil parameters assume that SPT was performed using a hammer system efficiency of 60 percent. If SPT is performed in borings for a project with a hammer system that has an efficiency “significantly” different (higher or lower) than 60%, published correlations are not applicable. Therefore, if the hammer system used for a project does not have an efficiency of 60 percent, the N-values must be corrected in order to compare them with published information, unless the specific published source indicates otherwise. Refer to Publication 222, Chapter 3.6.2, for requirements with respect to hammer system efficiency testing, assumed hammer efficiency values, and calculation of corrected N-values. Also, in the appendix of Publication 222, N_{60} values for various hammer types at indicated assumed efficiencies are provided.

5.3.4 SPT Depth/Overburden Pressure Correction

Another commonly used SPT N-value correction is for overburden correction. This correction was developed for sands due to the lack of cohesion that would provide resistance to lateral deformation in low overburden shallow depth conditions. The various corrections published do not differentiate for gradation, grain shape, or fines content. The correction also does not account for the stress history of a site. Correction for overburden can have a substantial and often unrealistic impact on N-values to a significant depth. In addition, relatively clean sand deposits are not common in Pennsylvania. The overburden correction is not intended to be used for fine grained soils. Because of the above reasons, correction of N-values for overburden pressure will not be considered for Department projects.

There are numerous other correction factors that are presented in published literature, including correction for borehole diameter, standard versus non-standard sampling methods, and

drill rod length. In general, as long as SPT is performed in accordance with Publication 222, Section 202 and ASTM D 1586, and based on typical practices used in Pennsylvania, it is not necessary to apply these additional factors for correction of field N-values.

5.4 ROCK CORE DRILLING

As discussed in [Chapter 3](#) of this publication, non-destructive rock core drilling is used for most subsurface explorations, particularly for structures and rock cuts. Using appropriate drilling methods and equipment is critical to obtaining representative rock core samples and information on rock quality and discontinuities. Minimum requirements for rock core drilling for Department projects are delineated in Publication 222, Section 204.

For most engineering analyses, the properties and behavior of rock masses are controlled by discontinuities in the rock mass, not the intact rock. Complete and accurate logging of core samples, including the discontinuities, in accordance with the requirements of Publication 222, Chapter 3.6.4 is important.

Field observations and measurements of discontinuities in an exposed natural rock slope provide the best opportunity for obtaining data on discontinuities in rock. Conversely, a core boring is limited by its size and orientation in providing information on discontinuities in rock masses. If a major discontinuity set orientation is steeply dipping, a vertical core boring can miss or encounter a limited number of discontinuities, even if the discontinuity spacing is moderate. Advancing the core boring at an angle may provide better discontinuity data.

The presence of voids or soil seams can control the engineering design of a structure or roadway. Voids can be due to carbonate bedrock or past underground mining. Evidence of voids or soil seams such as tool drops, loss of water return and decreases in drilling resistance should be logged along with the depth and thickness of the feature.

Borehole imaging with downhole cameras can be used to obtain a better understanding of subsurface voids. Technology is available for optical or acoustical imaging in vertical or angled boreholes that has the capability of providing measurements of size and orientation of discontinuities. This would be important on projects where the condition of discontinuities has a significant impact on design, such as a project with critical rock slope stability issues.

5.5 ESTIMATING ENGINEERING PROPERTIES OF SOIL AND ROCK

There are basically two options or methods for estimating soil and rock parameters to perform geotechnical analyses. One method is to use published information that correlates soil/rock parameters to basic soil/rock properties. The second method is to directly measure the parameter from field or laboratory testing. The method used is dependent upon several factors, including:

- **If the analyses are being performed for preliminary or final design.** During preliminary design, it is often necessary and more acceptable to use published correlations because site specific field and/or laboratory testing may not be available at that time. Also, preliminary analyses are often performed for

feasibility/estimation purposes and do not have to be as precise as when analyses are performed during final design.

- **The complexity of the analyses and the risk of the situation.** When the analysis being performed is not complex and the situation is of low or moderate risk, (e.g. a lightly loaded retaining wall or a spread footing founded on sand), using published correlations will often be appropriate. For more complex analyses and higher risk situations, (e.g. a heavily loaded bridge pier on any soil type or a spread footing founded on clay), direct measurements using field or laboratory testing (e.g., the internal friction angle from laboratory direct shear testing) would be more prudent than using published correlations.

5.5.1 Select Soil and Rock Parameters from Correlations

There are several published correlations that could be used to form a basis for the selection of soil and rock parameters. It is important that the correlation used have a track record of successful use on highway projects in Pennsylvania. The selected correlation is only as good as the data used to develop the correlation. The reliance or use of correlations to obtain soil and rock properties is justified and recommended in the following cases:

1. Specific data are simply not available and are only possible by indirectly comparing to other properties
2. A limited amount of data for the specific property of interest are available and the correlation can provide a complement to this limited data
3. The validity of certain data is in question and a comparison to previous test results allows the accuracy of the selected test to be assessed.

For soils, it is critical to understand that correlations have limitations, and they are not applicable to all soil types. For example, correlations for the shear strength of soil are typically and most reliably used for granular soils, and in particular, clean sands. These correlations become less reliable as the gravel and/or fines content in the sand increases. Shear strength correlations for fine grained soils are not reliable and must be used with caution. Furthermore, correlations are not available for all parameters, like those for estimating magnitude and the rate of consolidation settlement.

Correlations in general should never be used as a substitute for an adequate subsurface exploration program, but rather to complement and verify specific project-related information. Site specific laboratory data are always preferable to the exclusive use of correlations.

5.5.2 Select Soil and Rock Parameters Directly from Laboratory and/or Field Test Data

It is generally preferred to estimate soil and rock parameters directly from laboratory testing and/or in-situ field testing due to the variability of the properties that are estimated from correlations. Although testing generally provides the most accurate estimate, test results should not be used blindly without consideration of all available information. Due to the natural variability of soil and rock samples, the condition/disturbance of samples, and the potential for human error when performing the tests, test results will not always be representative of the in-

situ conditions. Additionally, when multiple tests for the same parameter are performed there will typically be some degree of variability between the results due to natural variations between the samples tested. Judgment must be used when reviewing the test results and other available information in order to select the parameters that will be used in the geotechnical analyses.

For soils, performance test results must generally be scrutinized more than index property test results because soil index property tests are not affected by as many factors as performance tests. For example, sample disturbance is not an issue when performing index tests, whereas sample disturbance will most likely, and sometimes severely, affect the results of performance tests. Additionally, performance test procedures are generally more complex/complicated compared to index tests; consequently, there is a greater possibility that performance test results could be flawed due to human error. Furthermore, unlike performance tests, index test results are not used directly in geotechnical analyses, but instead, are used to better estimate the variability of the subsurface and are used with published correlations.

Laboratory index test results for soils must be reviewed and compared with information on the Engineer's Boring Log. Test results that do not agree with field descriptions, moisture content, plasticity, etc., must be investigated. If the laboratory tested material is not available, other samples of similarly described materials must be reviewed to determine if the discrepancy is a result of the lower level of accuracy inherent in field logging, or possibly an error in the laboratory test results. Such errors could be as simple as samples being misidentified on the test reports, or more serious such as the test being performed improperly. If needed, additional testing should be performed to help resolve any discrepancies.

Performance test results for soils need to be closely scrutinized for reasonableness and consistency since these results are used directly in geotechnical analyses/calculations. As a first step, compare test results to available correlations and local experiences to evaluate if the results are generally reasonable. If the results fall outside the range of values that would be expected based on published correlations and local experiences for the soil type and in-situ condition, before using the test results there should be further evaluation of the test performance, sample disturbance, and stress history of the sample to establish a reasonable explanation for the discrepancy.

A detailed discussion on use of field and laboratory test results for selecting rock parameters is presented in [Section 5.5.4](#) of this publication.

Ideally, in order to evaluate test results for soil and rock, three or more tests should be performed on the same stratum/material for comparison purposes. When only one test is performed, it may not be possible to determine if the test result is representative of the stratum/material. Use of soil or rock parameters from only one test may result in an overly conservative design or conversely an inadequate design. When two tests are performed on the same soil stratum/material and the test results are consistent, they generally can be used with a fairly high level of confidence. However, if the test results vary considerably, it will be difficult to determine which result is more representative of the soil stratum. When results from three or more tests performed on the same stratum are available, there are two other methods to consider for selecting the parameters to be used for design:

- Evaluate the results obtained for the parameter being considered relative to all other information available for the soil stratum in question. If there is consistency for the other available information, an evaluation of all data can lead to an expected reasonable value or range of values for the parameter in question. If several of the conflicting test results are reflective and consistent with all other available information, then the parameter selection can be based on that portion of the test results. Depending upon the variability of the test results determined to be reasonably consistent with all other information, a simple average may be used to select the parameter value, or a statistical basis where one or two standard deviations is subtracted from the average of test results determined to be consistent with all other data. The degree of conservatism applied (simple average verses subtraction of one or two standard deviations) would be selected based upon an assessment of risk and cost. For example, the higher the assessed risk, the more conservative the parameter selection.
- In addition, it may be prudent to perform a sensitivity analysis by using a range of the values obtained from the laboratory tests to determine the influence of the results on the analyses. The sensitivity analysis may show that using a low/conservative value from the test results does not adversely impact the results/design recommendations.

It is often valuable to plot the results from all tests performed on similar stratum/materials onto one test report/graph. For example, when numerous tests are performed to estimate the drained shear strength of a soil, whether from direct or triaxial shear tests, it is helpful to show all the results on one Mohr's circle plot to better understand the variation of the results and to select a design value to use for analyses. This procedure can also be used with consolidation, gradation, unconfined compressive strength of rock and other types of test results.

It is understood that it is not always feasible or necessary to perform numerous tests on the same stratum/material. For example, one direct shear test may be appropriate for a project consisting of a small structure supported on a spread footing bearing on granular material. Since SPT N-values can be used with correlations to estimate the internal angle of friction of granular soil, the results from one direct shear test can be compared to published correlations to determine the reasonableness of the test result. However, this approach is not appropriate for fine-grained soil since published correlations are not reliable for these soils. Similarly, correlations are not reliable for estimating consolidation settlement parameters, so using one test result to select these parameters is problematic. For rock, as unconfined compressive strength values for intact rock are used directly for estimating parameters for the rock mass and the test is relatively inexpensive, multiple tests are recommended.

5.5.3 Engineering Properties of Soil

The selection of soil properties for design and analysis requires that the designer has a good understanding of the loading conditions and the soil behavior, has high quality soil sampling and testing, and has local geotechnical experience with the various geologic formations. This section provides guidance in the selection of engineering properties for cohesive and cohesionless soils for use in geotechnical design.

5.5.3.1 Soil Shear Strength

Shear strength parameters, which include cohesion and internal angle of friction, for soil are one of the most commonly needed parameters to perform geotechnical analyses, including: spread footing bearing resistance, deep foundation side and tip resistance, lateral earth pressure, slope stability, and more. Soil shear strength parameters are unique to each and every soil deposit, and the shear strength is influenced by many factors. The shear strength of granular soils is influenced by:

- Gradation (poorly graded, well graded, fines content, etc.)
- Shape of particles (rounded, angular, etc.)
- Unit Weight/Relative Density

The shear strength of fine grained soils is influenced by:

- Plasticity
- Consistency
- Nature of deposition (glacial lake, alluvium, colluvium, etc.)
- Stress history of soil deposit (underconsolidated, overconsolidated, etc.)

As discussed in [Chapter 4](#) of this publication, the shear strength parameters of granular/cohesionless soils are not dependent upon the presence of water since they are relatively free draining, and pore water pressure either does not develop in these types of soils, or it is minimal and only exists for a short period of time. However, the shear strength parameters of fine-grained, cohesive soil are dependent upon pore water pressure since these soils drain very slowly and can have pore water pressure present for extended periods of time. Consequently, the shear strength parameters of cohesive soils will vary depending upon the pore water pressure present in the soil.

5.5.3.1.1 In-Situ Granular/Cohesionless Soils

Recalling the shear strength equation, the soil parameters needed to calculate shear strength include cohesion (c) and internal angle of friction (ϕ).

$$s = c + \sigma' \tan \phi$$

where:

s = shear strength

c = cohesion

σ' = effective overburden stress

ϕ = internal angle of friction

Since no or minimal cohesion is typically present in coarse grained soils, this component of the shear strength equation is normally taken to be zero. Therefore, for granular soils, the only soil parameter that is typically needed to estimate shear strength is the internal angle of friction.

The internal angle of friction of granular soils can be estimated by correlating SPT N-values obtained from borings to published values of internal angle of friction. It can also be

estimated directly from laboratory test results. There are no in-situ tests that directly measure the internal angle of friction of granular soils.

As discussed in the [Chapter 4](#) of this publication, it is generally preferred and beneficial to estimate the internal angle of friction directly from laboratory shear tests. This provides a more accurate estimate of the internal angle of friction as compared to using published correlations. Using laboratory shear tests typically results in the geotechnical analyses and design recommendations being more representative of the actual field conditions, and often results in a more economical design.

Some circumstances do not permit or warrant laboratory shear strength testing of granular soil. For example, gravels are typically not practical to test due to grain-size limitations with standard laboratory testing equipment. Additionally, undisturbed samples of granular soils cannot generally be obtained, in which case remolded samples must be used. In some cases, these remolded samples may not be representative of the in-situ soil conditions, and therefore not useful/representative to test. Lastly, in some situations the internal angle of friction of the soil may not control the design of a foundation or slope, and a conservative internal angle of friction estimated from published correlations can be used without adversely impacting the geotechnical design recommendations or the cost associated with the recommendations.

There are numerous published correlations between SPT N-value and internal angle of friction for granular soils. Design Manual, Part 4 (DM-4), Table 10.8.3.5.2b-1P, provides correlations which are generally applicable to Pennsylvania, based on tables included in several textbooks by Joseph Bowles.

The original tables presented in Bowles' textbooks are based on N-values corrected for overburden pressure. However, as previously discussed, N-values obtained at depths up to 20 feet can increase significantly, and possibly unrealistically, when corrected for overburden pressure. Therefore, do not use the overburden correction at shallow depths for the estimation of the internal angle of friction with DM-4, Table 10.8.3.5.2b-1P (or AASHTO Table 10.4.6.2.4-1).

Similar to most if not all correlations, the correlation table in [Figure 5.5.3.1.1-1](#) requires consideration and judgment in order to select the internal angle of friction from the ranges given. For example, typically the following is true:

- Finer granular materials will fall at the lower end of the range
- Granular materials with fine-grained particles will reduce the internal angle of friction
- Coarse grained materials with little fines (i.e., say less than 5%) will fall at the upper end of the range
- Angular particles will have a higher internal angle of friction than rounded particles.

[Figure 5.5.3.1.1-1](#) on the following page presents a correlation of effective strength friction angle, ϕ' , of granular soils as a function of unit weight, soil type and relative density.

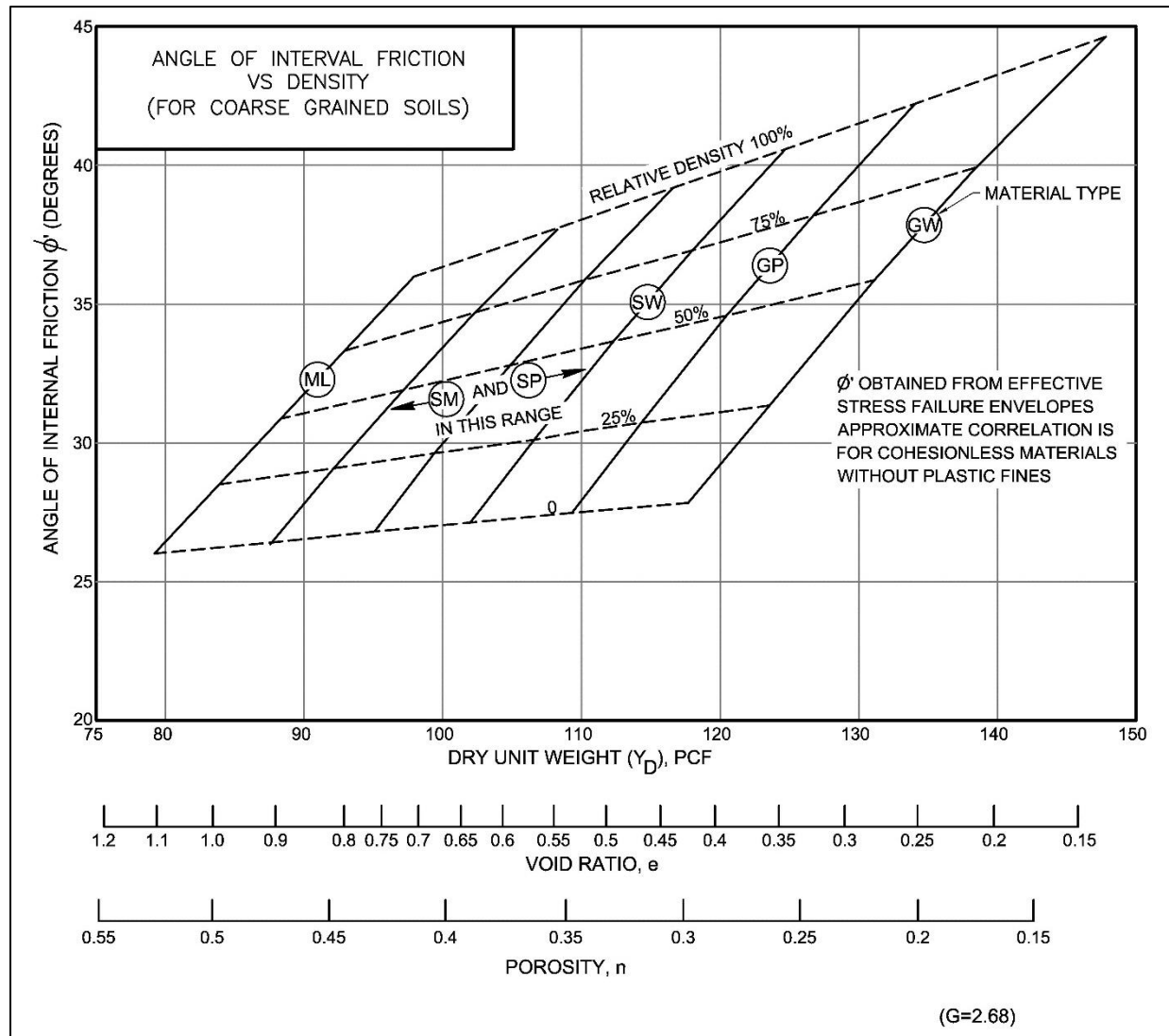


Figure 5.5.3.1.1-1 – Correlations of Effective Angle of Friction with Index Properties for Granular Soils

Notes: 1. Reference: NAVFAC DM 7.01, 1986

As previously discussed, care must be taken when using correlations when SPT is performed in strata containing coarse gravel, cobbles and boulders. These “large” materials can artificially inflate N-values (i.e., relative density) and result in an overestimation of the internal angle of friction.

Cone penetrometer testing (CPT) can also be used to estimate the internal angle of friction of granular soil, although CPT is mainly appropriate for sands since the presence of gravel can cause erroneously high results. Similar to SPT N-values, there are correlations between CPT tip resistance and internal angle of friction. If CPT data are obtained, it is recommended that FHWA Geotechnical Engineering Circular No. 5 be consulted for correlation to internal angle of friction.

5.5.3.1.2 In-Situ Fine-Grained/Cohesive Soils

Estimating the shear strength of cohesive soil is more complex than coarse grained soil, in part because it is affected by pore water pressure. When excess pore water pressure is present in the soil, the undrained shear strength will typically be comprised solely of cohesion (i.e., internal angle of friction of zero). However, when excess pore water pressure is not present, the drained (or effective) shear strength of the soil will commonly be comprised of only the internal angle of friction, although some component of cohesion may exist. Additionally, the shear strength of the soil can continually change as the excess pore water pressure in the soil dissipates. A more detailed discussion is provided in [Chapter 4](#) of this publication.

There are published correlations available for drained and undrained shear strength of cohesive soil. Examples are presented below. Additionally, unconfined compressive strength of cohesive soil can be measured in the field using a vane shear, and it can be estimated in the field or laboratory using a pocket penetrometer or hand torvane. However, **published shear strength correlations and hand penetrometer/torvane values are not to be used to determine final design shear strength values for cohesive soil.** Therefore, when the shear strength of cohesive soil is needed for geotechnical analyses, it is to be determined based on laboratory shear strength testing. Field vane shear tests may also be used to estimate the unconfined compressive strength for design; however, laboratory shear strength testing is still recommended even if field vane shear testing is performed.

The use of correlations and hand penetrometers/torvanes can only be used to estimate shear strength when laboratory testing is not possible, such as during the Alternatives Analysis or Preliminary Design Phase if borings have not been performed. When borings are obtained, shear strength testing of cohesive soil should be performed if it is needed for geotechnical analyses.

SPT N-values obtained in cohesive soils provide an indication of the consistency of the soil, but unlike granular soils, they are not useful for estimating the internal angle of friction for drained shear strength. Instead, the most commonly used correlation to estimate the internal angle of friction of cohesive soil utilizes the plasticity index (PI) determined from laboratory index testing (i.e., Atterberg limits test). This correlation is presented in [Figure 5.5.3.1.2-1](#) on the following page. As can be seen from this figure, the internal angle of friction values used to develop this correlation vary by nearly 10 degrees for a given PI.

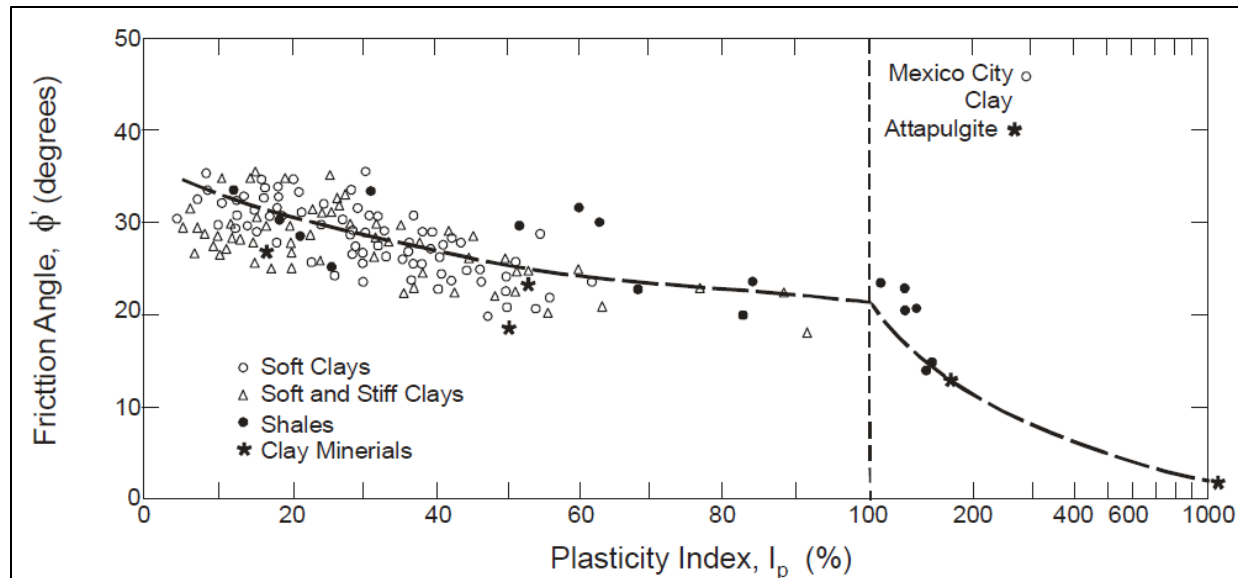


Figure 5.5.3.1.2-1 – Values of Friction Angle, ϕ' for Clays of Various Composition as Reflected in Plasticity Index¹

Notes: 1. Reference: Terzaghi, Peck and Mesri, 1996

The undrained shear strength of cohesive soil should not be estimated from correlations; however, when laboratory and/or field shear strength test data are not available, N_{60} -values can be used to estimate the undrained shear strength (i.e., cohesion). During undrained loading, the cohesion is assumed to equal one half of the unconfined compressive strength. Similar correlation tables are provided in NAVFAC DM-7.01 (1986) and “Foundation Analysis and Design”, second edition by Joseph Bowles (1977).

Table 5.5.3.1.2-1 – Correlation of SPT N_{60} Values to Unconfined Compressive Strength¹

N_{60}	Unconfined Compressive Strength (ksf)
<2	<0.5
2-4	0.5 - 1
4-8	1 - 2
8-15	2 - 4
15-30	4 - 8
>30	>8

Notes: 1. Modified after Peck, Hanson and Thornburn, 1974

Pocket penetrometer and/or hand torvane values should always be obtained by the drilling inspector to help estimate the cohesion of fine grained soils. These values should be compared with [Table 5.5.3.1.2-1](#), and judgment must be used to select a preliminary design value.

As stated above, correlation tables, pocket penetrometer readings, and hand torvane values can only be used for preliminary design. It is recommended that shear strength testing be performed during preliminary design if possible. Correlations should never be substituted for laboratory testing and any assumptions regarding shear strength should be verified by further testing during final design.

5.5.3.1.3 Peak or Residual Shear Strength

Shear strength values used for slope stability analyses must reflect the stress history of the stratum. Refer to Chapter 4, Sections [4.9.4](#) and [4.9.5](#) for a discussion of where peak and residual shear strength values should be used in design and appropriate laboratory tests. It is important that the need for residual shear strength values be identified prior to ordering laboratory testing so that the results will include residual shear strength values.

5.5.3.1.4 Use of Effective Cohesion in Engineering Analysis

Most soils in Pennsylvania exhibit both effective cohesion and friction, and sometimes drained shear strength test results (direct shear and triaxial) indicate that there is a cohesive component to the effective shear strength. The use of effective cohesion (c') in engineering analyses of drained (effective) strength conditions should be approached with caution.

Since the cohesive component of shear strength is water content dependent, and therefore variable, cohesion is often ignored in design. With continuing displacements, it is likely that c' will decrease to zero for long-term conditions, especially for highly plastic clays. A true effective cohesion value greater than zero would only be expected in cemented sands and some overconsolidated clays.

Compacted soils may exhibit effective cohesion. Compacted soils typically have an internal stress state between particles that exceeds the actual overburden stress. This excess pressure mimics an overconsolidated condition. The result of internal particle to particle stresses that exceed the normal stresses during testing is an effective cohesion. A granular, non-plastic soil that has been compacted to stress significantly exceeding the normal applied stress exhibits both c' and ϕ' during testing, even though the material is non-plastic. In this sense, compacting soil can be thought of as prestressing concrete.

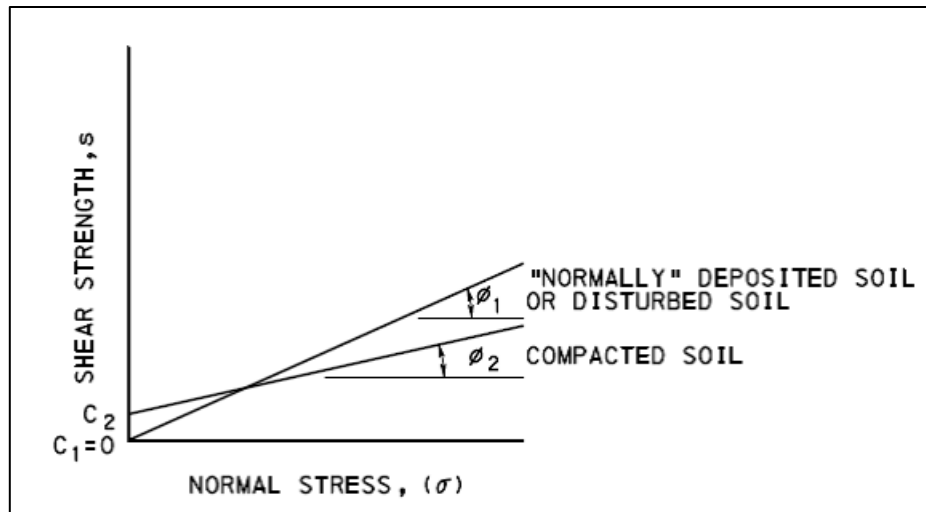


Figure 5.5.3.1.4-1 – Generalized Plots of Shear Strength

One engineering analysis where it may be appropriate to use a non-zero value for c' is when evaluating slope stability. When using a computer program such as SLIDE to search for the critical failure surface with $c' = 0$, the results often indicate that the critical failure surface is a shallow, sloughing type failure surface at the face of the slope. The frictional shear strength resisting movement is low because of the low normal forces near the face of the slope. Where shallow failure surfaces control, providing only a small amount of cohesion when running the analysis allows deeper, more significant failure planes to be evaluated.

However, if the material is not compacted or cannot be compacted in a manner that produces excess internal particle stresses (beyond applied overburden stresses), for example, with rock or open-graded, uniform aggregates, no effective cohesion is mobilized.

5.5.3.1.5 Shear Strength of Standard Embankment Materials

Soil shear strength of proposed embankments is commonly needed to perform geotechnical analyses, including slope stability, embankment foundation settlement, and lateral earth pressure. In most cases the “exact” materials that will be used to construct the proposed embankment(s) are not known during design as the material will often be obtained from a borrow source selected by the contractor. When material used to construct proposed embankments is known to be obtained from excavations, the material within the excavations is typically variable. Therefore, selecting representative soil parameters for design analyses is difficult. Publication 408, Section 206, defines several materials that can be used to construct embankments and backfills. These materials include:

1. Soil

Soil properties as mentioned in Publication 408, Section 206.2(a)1.a, are typical of the predominant soil types encountered in Pennsylvania, including silts and clays with varying amounts of sand and gravels, and sands and gravels with more than 35% fines (i.e., AASHTO Groups A-4, A-5, A-6 and A-7). Due to the wide variety of soil types that meet these Publication 408 properties, the range of possible soil parameters, and, in particular, shear

strength, is broad. Drained internal angle of friction (in degrees) could be as low as the mid 20's for silts and clays with little to no sand and gravel, and as high as the upper 30's for silty/clayey sands and gravels. Based on published information, historical data, and the performance of existing embankments, an internal friction angle of 30 degrees and no cohesion can be used as a starting point for proposed embankments when:

- The material used to construct the proposed embankment(s) will be obtained from an unknown borrow source selected by the contractor.
- The material used to construct the proposed embankment(s) will be obtained from a known source (i.e., on-site or off-site), but the material types from the source are variable, and no laboratory shear strength testing was performed on the proposed materials.
- The project specifications do not require a specific type of material (e.g., Granular Material, Rock, Select Borrow, etc.), or a material with a minimum internal angle of friction to be used to construct the embankment(s).

Where excavated material from the project will be available for project embankments, site specific boring and laboratory data should be used to select design parameters.

Although an internal angle of friction of 30 degrees may be high for some materials permitted in Publication 408, Section 206, these materials are required to be placed in compacted lifts, which will generally help improve shear strength. Furthermore, contractors normally avoid using fine-grained soils for constructing embankments, particularly embankments with significant height, because fine-grained soils are typically moisture sensitive, which can make them problematic to compact.

For slope stability analyses of new embankment slopes an internal angle of friction of 30 degrees and an effective cohesion of up to 70 psf may be used if the fines fraction of local soils typically has a plasticity index greater than or equal to 6.

On projects where the materials/sources that will be used to construct the proposed embankments are known, and the internal angle of friction of proposed embankments is needed for design purposes, laboratory shear strength testing should be performed. Samples of the material(s) should be remolded to the density and moisture content required by the embankment construction specification. This testing should permit the economical design of embankments, and avoid unnecessary embankment treatments, like flattened slopes, rock toe trenches, rock veneers, etc., that may result from designing with assumed/conservative soil parameters. Shear strength testing is most likely not warranted/necessary on projects where relatively low height embankments are proposed, slope stability is not a concern/issue, or proposed embankment material is known/predictable and soil parameters can be reasonably assumed.

Note that undrained shear strength parameters generally will not apply to the design of embankments on roadway projects. Material used to construct embankments must be at or near the optimum water content during placement/compaction; therefore, excess pore water pressure should not develop during construction. Additionally, the majority of embankments constructed will not become saturated during their service life. If saturation is possible/likely and this

condition requires analysis, it would be appropriate to analyze using the drained shear strength of the soil with an elevated water level to represent saturation.

2. Granular Material

Soils meeting the requirements presented in Publication 408, Section 206.2(a)1.b, are sands and gravels with less than 35% silt and clay (i.e., AASHTO Groups A-1, A-2, and A-3). Since these requirements are more restrictive compared to the Section 206 requirements for Soil, and since this material is predominantly granular, the range of the possible internal angle of friction is higher. The internal angle of friction (in degrees) of Granular Material compacted in accordance with Publication 408 most likely ranges from the low 30's to the low 40's. In the absence of specific laboratory shear strength testing data, specification requirements for the contractor to supply Granular Material with a specific minimum internal angle of friction must be provided that are consistent with shear strength parameters used for design. In such cases, an internal angle of friction of 32 degrees and no cohesion is a value that can be reasonable obtained for granular material; however, a higher value may be specified if necessary so long as material meeting the requirement is reasonably available. For AASHTO No. 8 or No. 57 Coarse Aggregate or AASHTO No. 2A or OGS Coarse Aggregate in accordance with the requirements of Publication 408, Section 703.2, an internal angle of friction of 34 degrees and no cohesion may be used. Do not apply effective cohesion within granular material for slope stability analyses.

3. Shale

Shale as described in Publication 408, Section 206.2(a)1.c, is a low strength material. Shale will typically break down when placed in lifts and compacted. Additionally, over time shale embankments will further degrade, particularly the outer portions that are exposed to water and air. Shale degrades into predominantly fine grained materials; therefore, a design internal angle of friction not exceeding that recommended for Soil (i.e., 30 degrees) will most likely be reasonable. Local experience, slake durability testing, and laboratory shear strength testing should be considered when shale is anticipated to be used to construct embankments and shear strength parameters are needed for design.

4. Rock

Rock embankment material is described in Publication 408, Section 206.2(a)1.d. The hardness and durability of rock embankment and the shape of the rock fragments will impact the internal angle of friction of the constructed embankment. Figure 10.4.6.2.4-1 in AASHTO provides correlations of friction angle with effective normal stress and rock type for gravels and rock fills. Typical ranges of values for rock embankment for use in Pennsylvania are as follows:

- Rock embankment consisting of quarry quality sandstone or limestone obtained from an approved source listed in Department Bulletin 14, the typical range for internal angle of friction is 40 to 45 degrees.
- Rock embankment consisting of sandstone, limestone or other durable rock with a slake durability index of greater than 90% with the appearance of the fragments retained on the drum classified as Type 1, as per ASTM D 4644, the typical range for internal angle of friction is 36 to 40 degrees.

- Rock embankment material that does not meet the rock type and durability requirements, but excludes slaking claystone, redbeds, and other forms of clay, silt, sand or mud, the typical range for internal angle of friction is 32 to 36 degrees.

Do not apply effective cohesion within rock embankment for slope stability analyses.

5. Random Material

Publication 408, Section 206.2(a)1.e, defines Random Material as any combination of the above classifications and may include concrete, brick, stone or masonry units from demolition. The presence of demolition material would not be expected to reduce the internal angle of friction of the predominant material classification. Therefore, the recommended internal angle of friction for the identified predominant material should be used.

5.5.3.2 Soil Unit Weight

Unit weight of soil and rock fill materials is also a parameter frequently needed to perform geotechnical analyses. Unit weights of soil and rock fill materials are not as variable and are influenced by fewer factors as compared to shear strength values. The factors that affect unit weight are primarily soil type/gradation, density/consistency and water content.

Note that the term density is commonly used in place of unit weight, which is technically correct. Unit weight is a measurement of force per unit volume (e.g., lb/ft³) that includes gravitational acceleration, whereas density is a measure of mass that does not include the effect of gravitational acceleration. Density (e.g., Mg/m³) multiplied by gravitational acceleration of 9.81 m²/sec will result in a force per unit volume (e.g., kN/m³) that is equivalent to unit weight (e.g., lb/ft³). Therefore, for the purpose of this publication, because the two terms are universally used interchangeably, the use of slugs in quantifying density is limited, and Department engineering publications use English units (except for some test methods), the two terms will be considered as equivalent in meaning.

Before discussing methods for estimating unit weight, it is important to discuss the various states that unit weight is presented, including dry, total (moist), and saturated unit weight.

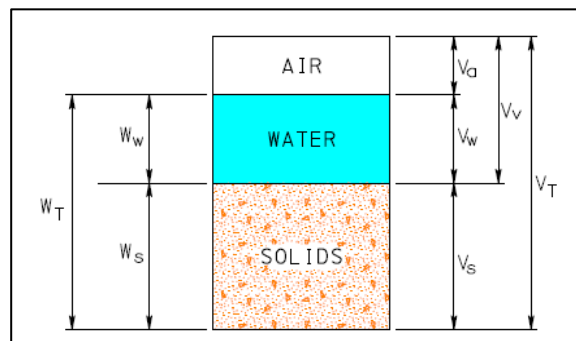


Figure 5.5.3.2-1 – Phase Diagram of Soil

The dry unit weight is determined by the following equation,

$$\gamma_d = W_s/V_t$$

where,

γ_d = dry unit weight

W_s = weight of the solid component of a soil sample

V_t = total volume of a soil sample

The moist, or total unit weight is determined by,

$$\gamma_t = W_t/V_t$$

where,

γ_t = moist unit weight

W_t = total weight including the water in the pores

The saturated unit weight (γ_s) is obtained when W_t is based on the state that all the pores are full of water.

The relationship between total and dry unit weight is as follows:

$$\gamma_d = \gamma_t/(1 + w)$$

where,

w = natural moisture content

Although laboratory testing can be performed on soils with a sufficient clay content to maintain form when removed from the sampler, published correlations such as the following may be used.

Table 5.5.3.2-1 – Moisture Content and Dry Unit Weight for Some Typical Soils in a Natural State¹

Type of Soil	Natural Moisture Content in a Saturated State, (%)	Dry Unit Weight, γ_d (pcf)
Loose uniform sand	30	92
Dense uniform sand	16	115
Loose angular-grained silty sand	25	102
Dense angular grained silty sand	15	121
Stiff clay	21	108
Soft clay	30-50	73-93
Loess	25	86
Soft organic clay	90-120	38-51
Glacial till	10	134

Notes: 1. Reference: Das, 2002

In Pennsylvania, if soil conditions are not extremely loose (soft) or dense (hard), typical total unit weight (γ_t) values fall between 110 pcf and 130 pcf. For compacted soil embankment from an unknown borrow source, a total unit weight value of 120 pcf is often assumed.

5.5.3.3 Soil Settlement

Settlements are often calculated based on results from in-situ tests and used either in empirical relationships or using equations from elasticity theory. The amount of settlement induced by the placement of load bearing elements on the ground surface or the construction of earthen embankments will affect the performance of the structure. The amount of settlement is a function of the increase in pore water pressure caused by the loading and the reduction of this pressure over time. The reduction in pore pressure and the rate of the reduction are a function of the permeability of the in-situ soil. All soils undergo immediate (elastic) and long-term (consolidation) settlement.

5.5.3.3.1 Immediate Settlement

Immediate settlement is calculated for all coarse-grained soils with large hydraulic conductivity and for dry, or slightly moist, fine-grained soils where significant amounts of pore water are not present. Elastic properties for evaluating immediate settlements are Elastic Modulus (E_s), and Poisson's Ratio (ν). Values of E_s and ν can be estimated from empirical relationships, in accordance with AASHTO Table C10.4.6.3-1, for both preliminary and final design.

If the evaluation of immediate settlement is critical to the selection or the design of the foundation type, in-situ methods such as Pressuremeter (PMT) or Dilatometer (DMT) test methods should be used to evaluate the stratum for final design.

Compacted fills constructed with good quality control in accordance with the material and compaction requirements of Publication 408, Section 206 should not experience significant immediate settlement.

5.5.3.3.2 Long-Term Settlement

Consolidation settlement (long-term) settlement analyses are used for all saturated or nearly saturated fine-grained soils.

For only preliminary design, the following correlations are helpful. If borings are done in preliminary design, undisturbed samples should be obtained and consolidation testing should be performed. Correlations should never be substituted for laboratory testing and any assumptions regarding consolidation parameters should be verified by further testing during final design.

The factors that affect consolidation settlement parameters include natural water content, composition (mainly mineralogy of the particles), structure, and hydraulic conductivity (permeability).

Many equations for correlating the Primary Compression Index, C_c , with index properties have been published. The following are some of the most useful correlations:

Table 5.5.3.3.2-1 – Correlations for C_c ¹

Correlation	Soil
$C_c = 1.56e_o + 0.0107$	All clays
$C_c = 0.007(LL - 7)$	Remolded clays
$C_c = 0.30(e_o - 0.27)$	Inorganic, cohesive soils: silt, some clay; silty clay; clay
$C_c = 0.009(LL - 10)$	Clay of medium to slight sensitivity ($S_t < 4, LL < 100$)
$C_c = 0.0115 w_n$	Organic soils, peat
$C_c = 0.75(e_o - 0.50)$	Low plasticity clays
Where: e_o = initial void ratio G_s = specific gravity PI = Plasticity Index LL = Liquid Limit S_t = sensitivity = (undisturbed undrained shear strength) / (remolded undrained shear strength) w_n = natural water content	

Notes: 1. Modified after Holtz and Kovacs, 1981

The ratio of the Recompression Index, C_r , to C_c typically ranges from 0.02 to 0.20 (Terzaghi, Peck and Mesri, 1996). For preliminary design, C_r can be assumed to be approximately 10% of C_c . Interpreting a C_r and C_c value for design should be based on a practical assessment of the data. The objective is to assign a value to each behaviorally different subsurface layer or to assign some representative value for the entire subsurface. Assessments to be made in evaluating compression data include depth ranges where the material is more silty or sandy as compared to other depth ranges, depth of transition from a crust layer to an underlying softer clay layer, and assessment of sampling disturbance.

Ratios of the Secondary Compression Index, C_α , to C_c for various geotechnical materials are shown in [Table 5.5.3.3.2-2](#). With the exceptions of peat, clays containing permeable layers, certain residual soils with high initial permeability and deposits where vertical drains are installed, secondary settlement is an insignificant component of total settlement.

Table 5.5.3.3.2-2 – Values of C_c / C_α for Geotechnical Materials¹

Material	C_c / C_α
Granular soils including rock fill	0.02 ± 0.01
Shale and mudstone	0.03 ± 0.01
Inorganic clays and silts	0.04 ± 0.01
Organic clays and silts	0.05 ± 0.01
Peat and muskeg	0.06 ± 0.01

Notes: 1. Reference: Terzaghi, Peck and Mesri, 1996

Values of the Coefficient of Vertical Consolidation, c_v , can vary widely because of the wide range of permeabilities that exist in soils. Approximate correlations of c_v with liquid limit are presented in [Figure 5.5.3.3.2-1](#).

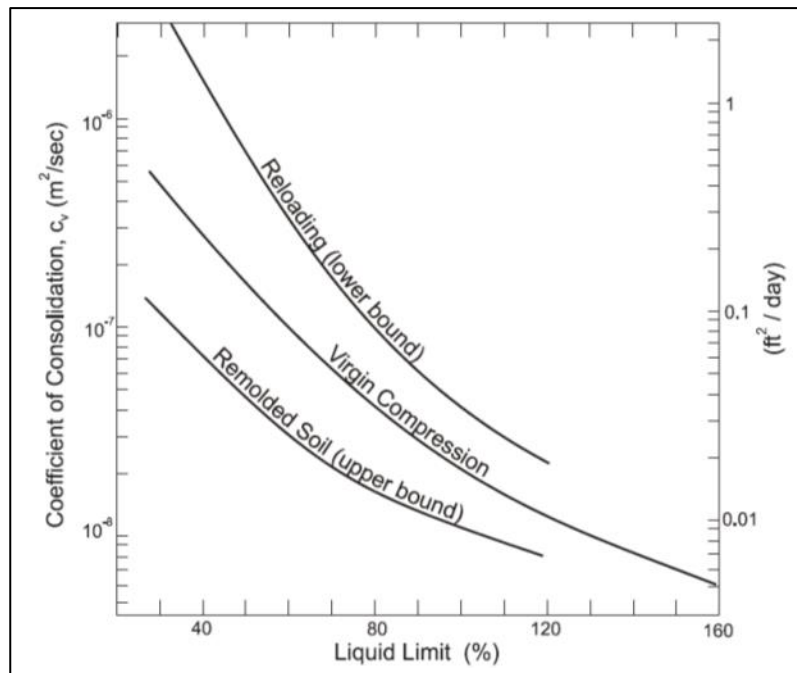


Figure 5.5.3.3.2-1 – Approximate Correlations between c_v and LL^{1, 2}

Notes: 1. NAVFAC DM-7.01, 1986

2. Reference: FHWA NHI-06-088, 2006

Where compacted fill is placed as embankment in wet areas or as structural backfill the use of fine grained soils should be avoided so that long term settlement is not a possibility.

5.5.3.4 Soil and Rock Shrink/Swell Factors

5.5.3.4.1 Earthwork Balance

Estimating shrink and swell factors for soil and rock excavated and placed as embankment for the purpose of earthwork balance evaluations is commonly based upon published data. Typically, the volume of soil in a compacted embankment is less than the volume of soil in-situ prior to excavation (shrinkage) and the volume of rock fill in an embankment is greater than the volume of rock in-situ prior to excavation. Possible sources for shrink/swell factors include the following:

- FHWA, Federal Land Highway Project Development and Design Manual, December, 2014, Chapter 6.
- Church, H.K., Excavation Handbook, McGraw-Hill, 1981.

Shrinkage factors utilized for soils in Pennsylvania generally range between 10 and 20 percent. Shrinkage factors on the low end of the range can be used for coarse-grained soils (A-1-a, A-1-b, and A-2-4). Shrinkage factors of 12 to 15 percent are more typical of soils with a larger fine-grained constituent (A-4, A-6). Shrinkage factors greater than 15 percent are appropriate only for soils with high organic content or fine-grained soils that derive from shale bedrock.

Soil shrinkage factors can be directly estimated by comparing the laboratory determined unit weight values of in-situ soils with the laboratory determined unit weight of the same material, compacted in accordance with project specifications.

Typically, the more massive and hard a rock formation is, the higher the potential swell factor. Published values for swell factors are higher than typically used in Pennsylvania. Historically, swell factors utilized for rock formations in Pennsylvania generally range from 0 to 20 percent. Swell factors for shale may range from 0 to 5 percent for soft shale, 8 to 10 percent for medium-hard to hard shale, and up to 10 to 15 percent for hard silty shale or siltstone. A swell factor of up to 20 percent may be used for good quality, thickly bedded sandstone and limestone.

5.5.3.4.2 Volume Change of In-Place Soils

Certain soil types (highly plastic) have a large potential for volumetric change depending on the moisture content of the soil. These soils can shrink with decreasing moisture or swell with increasing moisture causing distress to foundations and pavements. Although not usually an issue in Pennsylvania, the potential for volume change of in-place soils, whether existing or compacted embankment, should be considered for soils exhibiting a high degree of expansion as determined in [Table 5.5.3.4.2-1](#).

Table 5.5.3.4.2-1 – Determining Degree of Expansion in Soil

Degree of Expansion	Liquid Limit (LL)	Plasticity Index (PI)
High	>60	>35
Marginal	50-60	25-35
Low	<50	<25

Notes: 1. Reference: AASHTO T 258-81, Determining Expansive Soils

If a concern for shrink/swell behavior is identified for a site, the following laboratory methods can be used to estimate the volume change potential:

- AASHTO T 258-81, Determining Expansive Soils
- AASHTO T 92-97, Determining the Shrinkage Factors of Soils (withdrawn 2011 due to lack of use)
- ASTM D4506-08, One-Dimensional Swell or Collapse of Cohesive Soils.

Possible means to mitigate concerns for volume change potential include removal and replacement or lime stabilization

Compacted embankment soils meeting the requirements of Publication 408, Section 206 are unlikely to have a high potential for volume change unless the liquid limits and plasticity indexes are on the high end of ranges permitted by Section 206. A contractor would be unlikely to use such material due to the difficulty in obtaining the required moisture contents and compaction with such material. However, on a project with anticipated excavated materials that exhibit a marginal or high potential for volume change based on the above correlation, further evaluation of the potential for volume change would be justified.

5.5.3.4.3 Pyritic Soils and Rock

Pyritic soils and rock derived from the weathering of pyritic black shales and nearby non-black and non-carbonaceous shale strata commonly associated with coal bearing strata, are subject to swell and heaving. Pyritic soils, aggregates produced from pyritic rock and pyritic rock fill should not be used as fill or subgrade material below structures or pavements. Pyritic materials are also discussed in [Section 5.2.2.2](#) of this publication.

5.5.3.5 Soil Hydraulic Conductivity

Hydraulic conductivity (permeability) values are used in drainage and filter design, infiltration basin design, and retention basin design. The values are also helpful to Contractors in determining when and what type of a dewatering system will be needed for excavations.

Typical ranges of permeability are presented in the following tables:

Table 5.5.3.5-1 – Typical Permeability Values for Highway Materials^{1, 2}

Materials	Permeability (cm/sec)
Uniformly graded coarse aggregate	40 - 4×10^{-1}
Well-graded aggregate without fines	4×10^{-1} - 4×10^{-3}
Concrete sand, low dust content	7×10^{-2} - 7×10^{-4}
Concrete sand, high dust content	7×10^{-4} - 7×10^{-6}
Silty and clayey sands	10^{-5} - 10^{-7}
Compacted silt	7×10^{-6} - 7×10^{-8}
Compacted clay	Less than 10^{-7}
Bituminous concrete (new pavements) ³	4×10^{-3} - 4×10^{-6}
Portland cement concrete	Less than 10^{-8}

- Notes: 1. After Krebs and Walker, 1971
2. Reference: FHWA NHI-06-088, Vol. 1, 2006
3. Values as low as 10^{-8} have been reported for sealed, traffic compacted highway pavement

Table 5.5.3.5-2 – Typical Permeability Values in Soils^{1,2}

	10 ⁻¹¹	10 ⁻¹⁰	10 ⁻⁹	10 ⁻⁸	10 ⁻⁷	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³	10 ⁻²	10 ⁻¹	1
Coefficient of Permeability (m/s)												
Coefficient of Permeability (log scale)	10 ⁻⁹	10 ⁻⁸	10 ⁻⁷	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³	10 ⁻²	10 ⁻¹	1	10	100
	cm/s											
Permeability	Practically impermeable			Very low		Low		Medium		High		
Drainage conditions	Practically impermeable			Poor		Good						
Typical soil groups	GC → GM →			SM		SW →		GW →				
	CH			SC		SM-SC		SP →		GP →		
				MH								
				ML-CL								
Soil types	Homogeneous clays below the zone of weathering			Silts, fine sands, silty sands, glacial till, stratified clays				Clean sands, sand and gravel mixtures			Clean gravels	
				Fissured and weathered clays and clays modified by the effects of vegetation								

Note: The arrow adjacent to group classes indicates that permeability values can be greater than the typical value shown.

- Notes: 1. After Carter and Bentley, 1991
 2. Reference: FHWA NHI-06-088, Vol. 1, 2006

Permeability is impacted by void ratio, grain-size distribution, pore-size distribution, roughness of mineral particles, fluid viscosity, and degree of saturation. The permeability of coarse-grained soils may be significantly reduced by even small amounts of fine silt and clay sized particles. Compacted soils will have lower permeability than similar soil types in a loose condition. Stratified soils (alluvium, glacial lake) often have horizontal permeability that is many times greater than the vertical permeability. Also, the presence of confining layers with lower permeability will impact the drainage characteristics of a site.

Often the contrast in permeability between adjacent soils is more important than the absolute value. Correlations with soil classifications are often sufficient. For projects where drainage design is critical, field or laboratory tests should be used to evaluate the coefficient of

permeability. **Field infiltration testing in accordance with Pennsylvania DEP Best Management Practices (BMPs) is required for stormwater basin design.**

Pennsylvania DEP BMPs require that field testing be performed to determine if Infiltration BMPs are suitable at the site, and at what locations, and to obtain the required data for Infiltration BMP design. Requirements for site evaluation and soil testing can be found in Appendix C of the Pennsylvania Stormwater Best Management Practices Manual (PADEP, 2006).

5.5.3.6 Frost Susceptibility

Three conditions must be present in order for frost action in soils to occur:

- Freezing temperatures
- The presence of a water source to supply capillary moisture
- A frost-susceptible soil.

The formation of ice lenses is dependent on the grain size and pore size distribution. Typically, soils having silt as the fine-grained constituent are most susceptible to frost action. General guidelines for identifying frost-susceptible soils are presented in [Table 5.5.3.6-1](#).

Table 5.5.3.6-1 – Frost Susceptibility Classification of Soils^{1,2}

Frost Group	Degree of Frost Susceptibility	Type of Soil	Percentage Finer than 0.075 mm (#200) by wt.	Typical Soil Classification
F1	Negligible to low	Gravelly soils	3 - 10	GC,GP, GC-GM, GP-GM
F2	Low to medium	Gravelly soils	10 - 20	GM, GC-GM, GP-GM
		Sands	3 - 15	SW, SP, SM, SW-SM, SP-SM
F3	High	Gravelly soils	> 20	GM-GC
		Sands, except very fine Silty Sands	> 15	SM, SC
		Clays PI<12	-	CL, CH
F4	Very High	All Silts	-	ML, MH
		Very fine Silty Sands	> 15	SM
		Clays PI>12	-	CL, CL-ML
		Varied clays and other fine grained, banded sediments	-	CL, ML, SM, CH

- Notes: 1. After NCHRP 1-37A
 2. Reference: FHWA NHI-05-037, 2006

5.5.4 Select Rock Parameters

For most engineering analyses, the engineering behavior of rock masses are controlled by single or multiple discontinuities in the rock mass, not the strength of intact rock. Therefore, laboratory testing on intact rock cannot be applied directly to the determination of the rock mass parameters. Correlations and quality evaluations, i.e., Rock Mass Rating System (RMR) or Geologic Strength Index (GSI), rely heavily on visual observations and the resulting description of the rock core samples on the boring log.

Design analyses are typically required for the following foundations in rock:

- Bearing resistance for spread footings on rock.
- Bearing resistance for piles driven to absolute refusal on rock.
- Side and/or tip resistance for drilled in deep foundations (i.e., drilled shafts or micropiles) socketed into rock.
- Passive resistance for the portion of soldier piles below subgrade embedded in rock for non-gravity retaining structures.

For the above loading conditions, with the exception of drilled shafts or micropiles socketed into rock and subject to tension loads, the rock mass is in compression. The remainder of this discussion pertains to foundations in rock in compression. Tension is only permitted in the strength limit state for drilled shafts and micropiles socketed in rock, not at the service limit state.

For rock mass in compression, it is important that the rock mass is continuous, as evidenced by high core recovery values or borehole camera inspection. If the rock is not continuous, and exhibits low core recovery values, voids or soil seams, an assessment must be made to evaluate if there is sufficient rock mass present to adequately support the bearing loads, or if special treatments are required (i.e. lowering the foundation or grouting).

If piles are driven to absolute refusal on rock, the structural capacity of the pile controls the design. If the rock mass below the pile tip is continuous, and the presence of karst bedrock, mining or weak seams (coal, soil) has been accounted for, no further analysis of the bearing resistance in rock is required.

For the bearing resistance of spread footings and drilled in deep foundations, design methods incorporate the unconfined compressive strength of intact rock and an interpretation of rock quality, generally either Rock Quality Designation (RQD) or RMR. It is critical to understand when using these values, that both RQD and RMR were developed to assess rock mass behavior in tunneling operations. Loads on a rock mass, and the response of the rock mass in tunneling are very different than for bearing resistance situations. In tunneling situations, the rock is in tension and factors such as joint orientation and spacing, water or seepage pressures, and the condition of joints are extremely critical to the stability and strength of a rock mass.

When dealing with downward loads from foundations, the rock mass is in compression. Joint orientation and groundwater conditions are of low consequence. Joint spacing (weighted in

to the calculation of RQD) and joint condition are of significant importance. **Therefore, visual observations of RQD values and joint condition are very important to the design of foundations bearing on rock.** An exception, where joint orientation and water would be highly significant, would be for a spread footing on rock located above a rock cut as shown in [Figure 5.5.4-1](#).

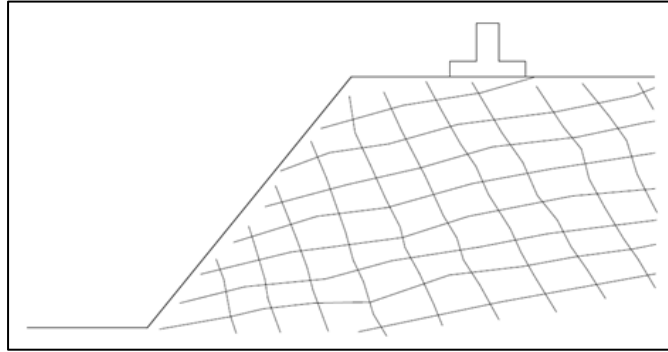


Figure 5.5.4-1 – Spread Footing on Rock above a Rock Cut

In this situation, the proximity of the footing to the exposed rock face will impact the level of confinement experienced by the rock mass. To assess if these factors need to be considered, the structure of the rock mass (strike and dip) should be measured and evaluated. If the plane of discontinuities on the exposed rock face intersect the ground surface, near, within or behind the footing then these factors are of significance and must be considered in evaluating bearing capacity.

It is important when using the semi-empirical design methods presented in DM-4, Section 10.6.3.2 that engineering judgement be applied when evaluating the results, particularly when the rock mass quality as defined by DM-4, Table 10.6.3.2.2-1P is “poor” or “very poor”. DM-4, Appendix B10P contains presumptive bearing values for rock developed for working stress design. While presumptive values should **not** be used for final design, they provide a “reality check” against which analysis results can be compared.

When rock is defined as “very poor” quality in accordance with DM-4, Table 10.6.3.2.2-1P, the ultimate bearing resistance for a spread footing should be calculated as an equivalent soil mass. This approach may also be appropriate for rock defined as “poor” on the table.

Determining the passive resistance and resulting embedment requirements for soldier pile retaining walls with the soldier piles drilled into rock requires the use of the shear strength of the fractured rock mass. Further discussion of this design case is provided in [Section 5.5.4.2](#) of this publication.

For the bearing resistance and passive resistance situations described above, the rock mass is usually in compression. For rock slope stability, the rock mass is in tension. Where the rock mass is fractured such that it contains a sufficient number of “randomly” oriented discontinuities such that it behaves as an isotropic mass, failure surfaces would intersect multiple discontinuities and the slope stability can be evaluated based on the shear strength of the

fractured rock mass. Evaluation of the shear strength of the fractured rock mass is discussed in [Section 5.5.4.2](#) of this publication.

Where the rock mass contains clearly defined dominant structural orientation such that it behaves as an anisotropic mass, failure surfaces would occur along a single discontinuity or set of discontinuities. The shear strength of a single discontinuity or set of discontinuities is based on the friction angle along that discontinuity. Evaluation of the friction angle along a discontinuity is further discussed in [Section 5.5.4.3](#) of this publication.

5.5.4.1 Rock Unconfined Compressive Strength

The laboratory test for Unconfined Compressive Strength of Intact Rock (ASTM D7012, Method C) is performed with loading in one direction, without lateral restraint. Unconfined Compressive Strength is also known as the Uniaxial Compressive Strength of Rock. An economical test, multiple rock unconfined compressive strength tests should be performed for almost any project that involves bedrock. Laboratory test results can be compared to the typical ranges such as provided in DM-4 Table 10.6.3.2.2-2 to evaluate if they are reasonable.

Rock core samples chosen for testing should have a length-to-diameter ratio (L/D) of 2.0 to 2.5. When samples of rock core with the required L/D are not available, samples with an L/D as close to 2.0 may be used as discussed in [Chapter 4, Section 4.9.8](#).

Point-load strength testing can be performed on rock specimens in the form of core, cut blocks, or irregular lumps. It can be performed on core samples with an L/D less than 2.0 to complement the results of the unconfined compressive strength testing. It is **not** appropriate for weak rocks with unconfined compressive strength less than approximately 500 ksf. It can be performed in the field with portable equipment or in the laboratory.

Point-load strength testing is **not** a direct measurement of strength. It provides an index of the unconfined compressive strength of intact rock. On the average, unconfined compressive strength of rock is about 20 to 25 times the point-load strength index. A value of 24 is commonly used. However, tests on many different types of rock show that the ratio can vary between 15 and 50. **Therefore, point-load tests should only be used in conjunction with unconfined compressive strength tests for correlation.**

5.5.4.2 Rock Mass Shear Strength

AASHTO, Section 10.4.6.4, contains recommendations and methodology for estimating rock mass strength. The methodology involves first classifying the rock mass using the Geologic Strength Index for Jointed Rocks (GSI) and then assessing the strength of the fractured rock mass using the Hoek Brown Failure Criterion.

The GSI system was developed only for the function of estimating rock mass properties. It has no rock reinforcement or support design capacity. The GSI is based on an assessment of the lithology, structure and condition of discontinuity surfaces in the rock mass and is estimated

from visual examination of the rock mass exposed in outcrops, cut slopes and borehole cores. In most cases, the GSI is best described by assigning it a range of values.

The determination of the strength of a rock mass using the GSI and the equations of the Hoek Brown Failure Criterion presented in AASHTO is only applicable to a fractured rock mass of “randomly” oriented discontinuities, such that it behaves as an isotropic mass. Stated another way, the behavior of the rock mass can be expected to be largely independent of the direction of the load. These methods should not be applied to rock masses with well-defined dominant structural orientation, where the stability of a slope or excavation will be controlled by the three-dimensional geometry of the intersecting discontinuities and the free faces of the slope or excavation.

In practice, for most rock slope stability applications, the use of the GSI and Hoek Brown Failure Criterion for estimating rock mass shear strength may be limited. Most rock formations for which steepened excavation slopes are considered fall into the category of structurally controlled stability and should be approached based on the shear strength of the discontinuity or set of discontinuities in accordance with [Chapter 8](#) of this publication.

For the design case of retaining walls and temporary support of excavation utilizing soldier piles, the determination of the passive resistance for the portion of the soldier pile embedded in rock below the excavation subgrade does require use of the shear strength of the rock mass. Design values of the shear strength of the rock mass should be determined using GSI classification and the Hoek Brown Failure Criterion in accordance with AASHTO, Section 10.4.6.4. Because the GSI is estimated based on observations of outcrops, cut slopes and borehole core samples, which may be limited, it is recommended that a range of GSI values be assumed and the impact to the design value of shear strength evaluated.

5.5.4.3 Rock Joint Friction

When the failure mode for a rock slope or underground excavation is controlled by a single discontinuity or set of discontinuities, the shear strength is represented by the friction angle along that discontinuity. Direct shear tests can be performed on core samples with sawn or ground rock surfaces. Typical values of friction angles for smooth joints for various rock types are presented in AASHTO, Table C10.4.6.4-1.

Because a natural discontinuity in rock is not smooth like a sawn or ground surface, the friction angle for the smooth joint can be adjusted to account for roughness. If the major discontinuity or set of discontinuities contain infilling, the shear strength of the discontinuity will be influenced by the thickness and properties of the infilling. Methodologies for evaluating the shear strength of a discontinuity with surface roughness and/or infilling are presented in FHWA GEC 5, Evaluation of Soil and Rock Properties (2002).

5.5.4.4 Rock Elastic Modulus

Elastic settlement of foundations bearing on continuous rock is rarely a concern for highway structures unless foundations are subjected to very large loads or where settlement tolerance may be small. Therefore, in most cases published data may be used to estimate the elastic modulus of intact rock (E_R). AASHTO Table C.10.4.6.5-1 contains ranges of E_R for various rock types. The elastic modulus of the rock mass (E_m) can be estimated based on the determined value or range of values for GSI and the elastic modulus of intact rock (E_R) in accordance with AASHTO Table 10.4 6.5-1. The AASHTO table is reproduced below displaying the equations for English units rather than metric as shown in the original. Note that if soil-filled discontinuities or voids in the rock below the foundation are present, they will have a greater impact on anticipated settlement than the stiffness of the rock mass.

Table 5.5.4.4-1 – Estimation of E_m Based on GSI¹

Expression ^{2, 3, 4, 5}	Remarks	Reference
$E_m = \left(q_u \left(\frac{6.89}{100} \right) \right)^{0.5} \left(10^{\left(3 + \frac{GSI-10}{40} \right)} \right)$ <p style="text-align: center;"><i>for $q_u \leq 14.5$ ksi</i></p>	Accounts for rocks with $q_u < 14.5$ ksi	Hoek and Brown (1997), Hoek et al. (2002)
$E_m = 145.14 \left(10^{\left(\frac{GSI-10}{40} \right)} \right)$ <p style="text-align: center;"><i>for $q_u > 14.5$ ksi</i></p>		
$E_m = \left(\frac{E_R}{100} \right) e^{GSI/21.7}$	Reduction factor on intact modulus, based on GSI	Yang (2006)

- Notes: 1. Modified from AASHTO (2014) Table C10.4.6.5-1 to English units
2. E_m = Equivalent Rock Mass Modulus (ksi)
3. q_u = Uniaxial Compressive Strength of Intact Rock (ksi) = C_o in DM-4, Table 10.6.3.2.2-2P (2015)
4. GSI = Geological Strength Index
5. E_R = Modulus of Intact Rock

5.5.4.5 Slake Durability

Pennsylvania contains some sedimentary rock types that degrade quickly when exposed to the elements or are disturbed. These types of rock are claystones, mudstones and shales. Sometimes siltstones are a concern as they may be misclassified mudstone. While these rocks are underground and confined, they maintain their strength, even when saturated. Once exposed

or disturbed, water can cause rapid degradation. This degradation is known as slaking. Slaking results in the rock quickly degrading back into soil.

Slaking behavior impacts several geotechnical designs, including the following:

- Rock cut slopes, including temporary excavation slope can be de-stabilized when deterioration of slaking strata causes undercutting of more durable rock, creating an unstable condition.
- The subgrade of spread footings founded on slaking rock can quickly deteriorate when exposed. Consider protecting the footing subgrade with a mudslab while the excavation is open.
- A pile driven into slaking rock may not maintain its resistance because the disturbance and breakage caused by driving the pile reduces the confinement. Past experience in Allegheny County has shown long-term settlement of bearing piles founded in claystone material. It may be necessary to drill through slaking rock and anchor the pile in more durable rock or to use a drilled shaft filled with concrete to provide the needed confinement. Another approach is to design the pile as a friction or end bearing pile and drive to capacity using wave equation and dynamic monitoring rather than to 20 bpi refusal. In accordance with DM-4, 10.4.7.2.4P, piles should not be terminated in claystone unless a slake test has been performed that indicates that the claystone is not subject to deterioration.
- Where excavated rock is considered for re-use as an embankment fill (rock fill) potential slaking behavior would result in strength loss and settlement. This can be mitigated by placing in thinner lifts than typically required for rock fill and performing vibratory compaction to mechanically break down larger pieces of rock. Even if mitigation measures described above are specified, slaking rock is not to be used to construct embankments with slopes steeper than 2H:1V.

Rock slake durability is discussed in more detail in [Chapter 15](#) of this publication.

5.6 FINAL SELECTION OF DESIGN VALUES

The development of soil and rock properties for geotechnical design purposes begins with developing/defining the geologic strata present at the site in question. Therefore, the focus of geotechnical design property assessment and final selection is on the individual geologic strata identified at the project site. A geologic stratum is characterized as having the same geologic depositional history, stress history, degree of disturbance, and similarities throughout the stratum in density, source material, stress history, hydrogeology, and macrostructure. All the information that has been obtained up to this point including preliminary office and field reconnaissance, boring logs, laboratory data, etc. are used to determine soil and rock engineering properties of interest and develop a subsurface model of the site to be used for design. Data from different sources of field and lab tests, from site geological characterization of the site subsurface conditions, from visual observations obtained from the site reconnaissance, and from historical experience with the subsurface conditions at or near the site must be combined to determine the engineering properties for the various geologic units encountered throughout the site. Soil and rock properties for design should not be averaged across multiple strata, since the focus of this

property characterization is on the individual geologic stratum. Often, results from a single test (e.g. SPT N-values) may show significant scatter across a site for a given soil/rock unit. Perhaps data obtained from a particular soil unit for a specific property from two different tests (e.g. field vane shear tests and lab UU tests) do not agree. The reasons for the differences shall be evaluated, poor data eliminated, and trends in data identified.

Recognizing the variability discussed in the previous paragraph, depending on the amount of variability estimated or measured, the potential impact of that variability (or uncertainty) on the level of safety in the design should be assessed. If the impact of this uncertainty is likely to be significant, more data could be obtained to help reduce the uncertainty. Since the sources of data that could be considered may include both measured laboratory data, field test data, performance data, and other previous experience with the geologic unit(s) in question, it will not be possible to statistically combine all this data together to determine the most likely property value. Engineering judgment, will be needed to make this final assessment and design property determination. At that point, a decision must be made as to whether the final design value selected should reflect the interpreted average value for the property, or a value that is somewhere between the most likely average value and the most conservative estimate of the property. However, the desire for design safety must be balanced with the cost effectiveness and constructability of the design.

Although laboratory testing is often considered to be “expensive”, and is often considered an item that can be used sparingly in an attempt to reduce design costs, these are generally not realistic points of view. Consider the construction cost savings (i.e., excavation, concrete, temporary excavation support, etc.) that can be realized from reducing the width of a spread footing by only one or two feet as a result of using a slightly higher internal angle of friction based on good quality direct shear testing instead of using lower/more conservative value in the absence of laboratory data. With laboratory strength test results, the design may be performed using a higher resistance factor which may also reduce the footing size. Also, consider the construction cost savings (i.e., need for wick drains/sand columns, appropriate number of wick drains/sand columns, reduced or no time for surcharge/waiting period, settlement monitoring requirements, etc.) from having more refined estimates of consolidation settlement magnitude and time resulting from good quality laboratory testing data versus using more conservative parameters in the absence of laboratory data. Furthermore, consider the construction costs resulting from change orders, claims and/or reduced structure service life associated with designs using assumed parameters selected from correlations instead of from laboratory testing in order to save a relatively small amount during design.

Design values that are more conservative than the mean may still be appropriate, especially if there is an unusual amount of uncertainty in the assessment of the design properties due, for example, to highly variable site conditions, lack of high quality data to assess property values, or due to widely divergent property values from the different methods used to assess properties within a given geologic unit. Depending on the availability of soil or rock property data and the variability of the geologic strata under consideration, it may not be possible to reliably estimate the average value of the properties needed for design. In such cases, the DGE may have no choice but to use a more conservative selection of design parameters to mitigate the additional risks created by potential variability of relevant data.

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CHAPTER 6 – SOIL SLOPE STABILITY

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CHAPTER 7 – SETTLEMENT

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2018
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 8 – ROCK CUT SLOPE AND CATCHMENT DESIGN

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8.1 INTRODUCTION

This chapter of the publication provides guidelines, recommendations, and considerations for the design and construction of new rock cut slopes, the rehabilitation of existing rock cut slopes, and rockfall mitigation and control measures for Department projects. This chapter has been formulated based on review and analysis of the latest Department practices and available published literature. This chapter also discusses the methodology to prepare designs while appropriately considering the proposed construction, geologic hazards, slope constructability, construction costs, and public safety. The guidelines and considerations presented in this chapter should be used during project reconnaissance, evaluation/design, maintenance of rock cut slope/catchment areas and design review for projects involving rock cut slopes.

8.1.1 Purpose

The primary purpose of rock cut slope and catchment design is to provide the Designer with efficient and consistent guidance for the design of new rock cut slopes, mitigation and/or the rehabilitation of existing rock cut slopes, and the catchment and control of rockfall. The Designer is responsible for preparing a design that is based on site-specific geotechnical exploration and achieves the optimal balance of safety, construction costs, and future maintenance costs.

8.1.2 Geologic History

Pennsylvania exhibits a diverse range of geological and structural conditions across the Commonwealth as observed in exposed geologic formations and structural features (faults and folds) in many existing roadway cuts. The variability within the formations and their features is principally due to historic geologic activity involving four mountain-building events (e.g., Grenville, Taconic, Acadian, and Alleghenian) each followed by periods of erosion and deposition. These geologic processes have produced igneous, metamorphic, and sedimentary rock types and resulted in varying degrees of rock deformation.

8.1.3 Physiography

A total of six physiographic provinces exist within the Commonwealth. Each province possesses different geologic and structural features which complicate the development of a statewide set of prescribed cut slope geometric design recommendations for a given set of geologic conditions. For example, in the western and northern portions of the Commonwealth, and more specifically the Appalachian Plateaus and Central Lowlands physiographic provinces (refer to [Figure 8.1.3-1](#)), the geology is largely dominated by horizontal to near horizontal sequences of interbedded sedimentary rock (shale, sandstone, limestone, coal, and siltstone). This has resulted in a variably wide range of rock strengths and vertical rock durability within an individual rock cut. Conversely, in the central and eastern portions of the Commonwealth, and more specifically the Ridge and Valley, Piedmont, and New England physiographic provinces, the geology is largely dominated by a non-horizontal or inclined structural fabric comprised of sedimentary, igneous, and metamorphic rock. These structural and lithological differences profoundly affect potential rock slope failure modes, and therefore must be factored into the

design for rock cut slopes. Based on the lithologic and geologic structural differences, the Commonwealth is divided into two regions as shown in [Figure 8.1.3-1](#).

Region I includes all of PennDOT Districts 1, 10, 11, and 12; northern and western portions of District 2; northern portions of Districts 3, 4, and 5; and the western portion of District 9. Region II includes all of PennDOT Districts 6 and 8; the southeastern portion of District 2; southern portions of Districts 3, 4, and 5; and the eastern portion of District 9.

[Table 8.1.3-1](#) lists selected properties of rock masses found in five physiographic provinces, which are grouped into Regions I and II. The Atlantic Coastal Plain province of southeastern Pennsylvania has been omitted since it is underlain by unconsolidated to poorly consolidated sands and gravels. These geologic properties warrant different design guidelines for the design/mitigation of rock cut slopes based on physiographic location.

Table 8.1.3-1 – Rock Characteristics of Pennsylvania’s Physiographic Provinces¹

Geologic Characteristic	REGION I	REGION II
	Appalachian Plateaus and Central Lowlands Provinces	Ridge and Valley, Piedmont, and New England Provinces
Bedding or Foliation	Horizontal to Near Horizontal Bedding, typically less than 30°	Bedding or Foliation highly variable; beds may be overturned or part of a thrust sheet due to faulting
Bedrock Durability	Highly variable - significant shale and claystone units susceptible to slaking and rapid disintegration	Less variable - fewer weak shales and claystones; generally, more crystalline rocks, resistant sandstones, and solution-prone carbonate rocks
Joints/Discontinuities	Joints are predominantly vertical to near vertical, typically between 70°-90° and usually occur in a regular pattern; with two nearly orthogonal tectonic joints and a near vertical valley stress relief joint	Joint orientations are highly variable and may occur in an irregular pattern; cleavage planes may be highly developed; variety of faults, which may be common
Failure Modes	Predominantly undercutting due to differential weathering, which triggers wedge failures and rockfalls; red beds (claystones) of western PA are abnormally susceptible to landsliding	Planar, wedge, and toppling failures; undercutting is less prevalent
Global Stability Failure	Low to moderate concern	High concern

Note: 1. This table provides a general overview of anticipated geologic conditions within the Commonwealth and serves as a starting point for the design/mitigation of rock cut slopes, however some projects may not strictly align with these geologic characteristics and failure modes.

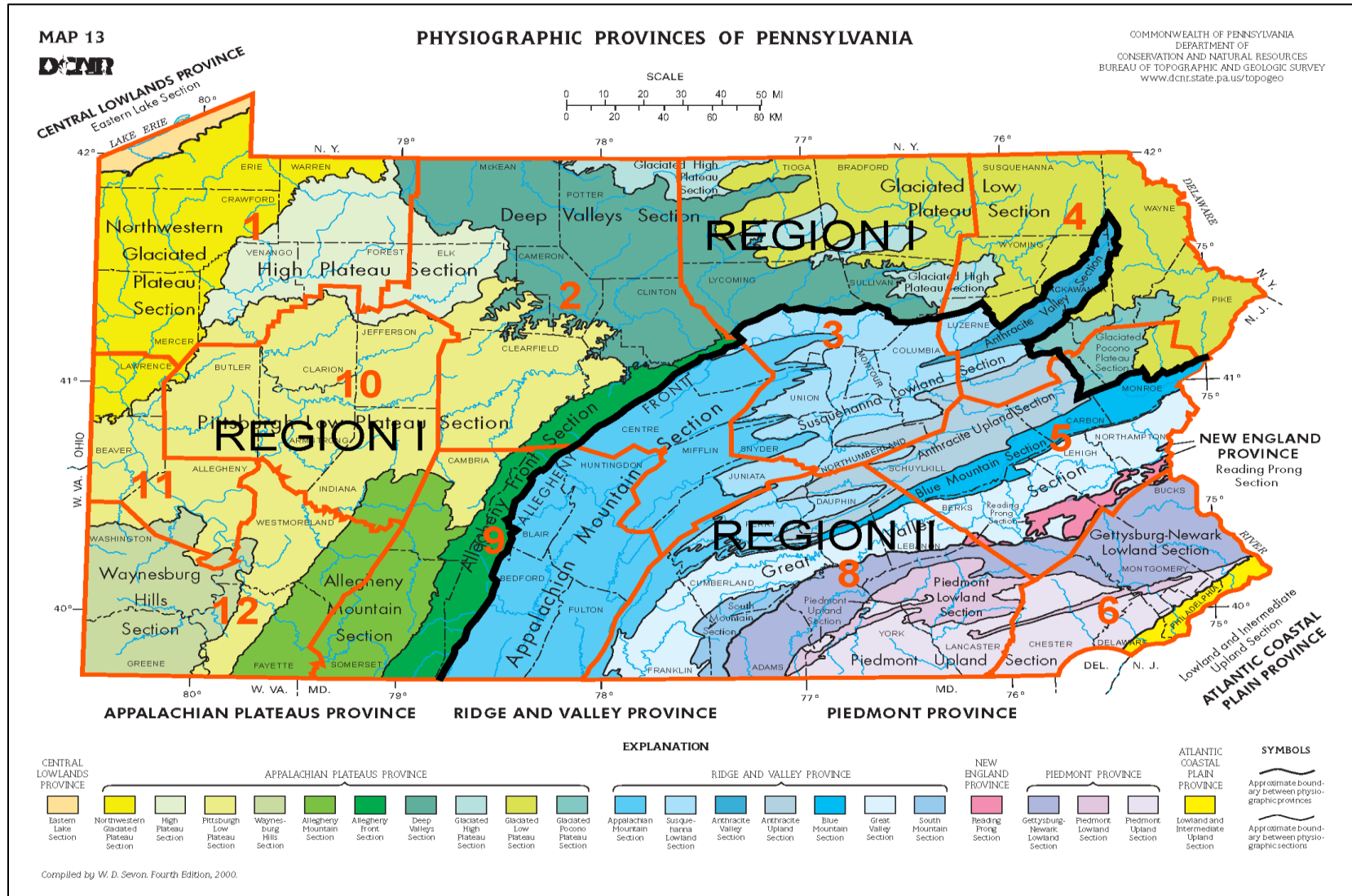


Figure 8.1.3-1 – Physiographic Province Map of Pennsylvania, with PennDOT Districts¹

Notes: 1. Adapted from Reference: Sevon, 2000

8.2 ROCK CUT SLOPE TERMS AND DEFINITIONS

The following terms and their definitions/descriptions are provided so a common nomenclature can be established for the rock cut slope(s) within a given project area. These are presented to aid in the understanding of this chapter and applicable to the guidance and recommendations contained herein. These terms cannot define every geologic setting or condition that will be encountered and the Designer and District Geotechnical Engineer (DGE) must have a shared understanding of the geologic setting, limitations, and acceptable risk for the design and/or mitigation of cut slopes for any given project. Also, note that while a given material may fit particular definitions for descriptions, the highly complex and variable conditions frequently exhibited by rock masses may dictate the treatment of a material outside of the indicated descriptions.

1. **Lithologic Unit** – A body of rock comprised of a similar mineral composition, grain size, and geotechnical engineering characteristics (e.g., shear strength, durability, degree of weathering).
 - **Durable Lithologic Unit** – A lithologic unit that is based on the following:
 - Metamorphic, or igneous rock units, or sedimentary rock units consisting of limestone/dolomite, or sandstone, having visual hardness description of “medium hard” to “very hard” in accordance with Publication 222, Chapter 3.6.4(f).
 - Metamorphic, igneous rock, or sedimentary rock consisting of limestone or sandstone, having a visual hardness description of “very soft” to “soft” in accordance with Publication 222, Chapter 3.6.4(f), and having a second cycle (Id₂) Slake Durability Index (SDI) value of 85 percent or greater in accordance with ASTM D4644.
 - Any siltstone with a second cycle (Id₂) SDI greater than 85 percent in accordance with ASTM D4644.
 - **Non-Durable Lithologic Unit** – A lithologic unit described as a shale, claystone, or a lithologic unit described as “very soft” to “soft” in accordance with Publication 222, Chapter 3.6.4(f), with a unit weight less than 140 pcf, and with an Id₂ SDI value less than 85 percent.
2. **Design Section** – A portion of a slope, or the entire slope, that can be cut at a consistent angle. A design section may be comprised of single or multiple lithologic units. A design section can be selected based on characteristic lithology and the anticipated slope failure(s). The thickness of a design section can range from a relatively short thickness (minimum 10 feet) to the height of the entire slope. Three (3) types of design sections are considered for rock cut slope design, defined as follows:

- **Competent Design Section** – Consists of greater than 90 percent Durable Lithologic Units. The failures anticipated to occur in this design unit are those controlled by unfavorable orientation of discontinuities (plane, wedge, or toppling failures) and hydrostatic forces. An example of a rock cut meeting the requirements for a competent design section is presented in [Figure 8.2-1](#).



Figure 8.2-1 – Competent Design Section comprised of Durable Lithologic Units

- **Incompetent Design Section** – Consists of greater than 90 percent Non-Durable Lithologic Units. The failures anticipated in this design section include raveling and rotational slides. An example rock cut meeting the requirements for an incompetent design section is presented in [Figure 8.2-2](#).



Figure 8.2-2 – Incompetent Design Section comprised of Non-Durable Lithologic Units

- **Interlayered Design Section** – Consists of interlayered Durable and Non-Durable Lithologic Units, each ranging in proportion from 10 percent to 90 percent. Failures induced from erosional undercutting and freeze thaw are the anticipated primary failure modes in this design section. However, complex slump and translational slides are possible depending on the degree of weathering and discontinuity conditions. An example rock cut meeting the requirements for an interlayered design section is presented in [Figure 8.2-3](#).



Figure 8.2-3 – Interlayered Design Section comprised of both Durable and Non-durable Lithologic Unit

3. **Discontinuity** – Discontinuities (e.g., joints, fractures, shears, faults, etc.) within a rock cut slope present planes of weakness where sliding can occur. The Designer should be aware of and understand discontinuity engineering properties such as orientation, spacing, continuous length of the discontinuity, roughness, durability, weathering, and infilling. Reference the revised 3rd Edition of “Rock Slope Engineering” by Hoek and Bray (1980) for an overview of the relationship between discontinuities and rock slope stability. Reference Publication 222, Chapter 3.6.5 for a discussion on discontinuities and descriptors within a rock structure.
4. **General Structural Geology and Rock Slope Stability Terms** – The Designer should understand the following structural geologic terms when using this guidance Chapter.
 - Line of Strike – Direction of the line that is formed by the intersection of a planar feature with the horizontal surface (Figure 5). Strike is typically expressed as degrees east or west of north or south (e.g., N25°E).
 - Angle of Dip – Angle between a planar feature and a horizontal surface measured in a vertical plane normal to the strike direction ([Figure 8.2-4](#)).

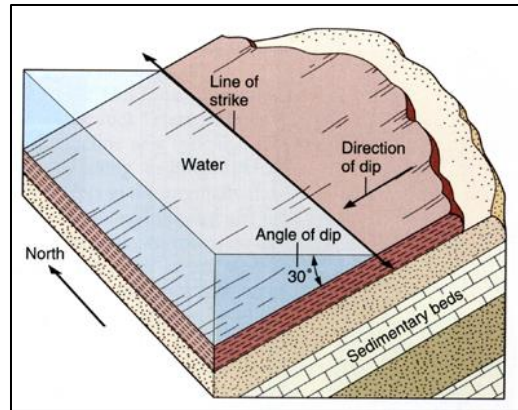


Figure 8.2-4 – Strike, Dip, and Dip Direction Diagram

- Direction of Dip – Azimuth in the down dip direction of the vertical plane normal to the strike.
 - Kinematic Analysis – Is a method used to analyze the structural fabric of the rock mass to determine if the orientation of the discontinuities could result in instability of the slope under consideration. Determination of kinematically possible modes of failure (planar, wedge, and toppling failures) is accomplished by means of a stereographic analysis.
 - Limit Equilibrium – Upon determination of a kinematically possible mode(s) of failure, a quantitative limit equilibrium stability analysis is performed to compare the forces resisting failure with the forces causing failure resulting in a calculated factor of safety.
5. **Simple Rock Cut Slope Design** – A proposed rock cut slope requiring limited design effort based on an evaluation of the local geological conditions, roadway and slope geometry, and proposed construction. The following general conditions are characteristics representative of Simple Rock Cut Slope Designs:
- Low risk to traveling public, Department facilities, and/or service interruption.
 - Maximum height of the proposed rock cut is less than 20 feet.
 - Predominant sliding plane discontinuities dip 20 degrees or less out of the proposed cut slope face.
 - The strike (the direction or bearing 90 degrees from the direction of maximum dip) of the discontinuities must not be within 20 degrees of the strike of the slope face (refer to general structural geology terms for explanation of terms).
 - Preliminary observation indicates global stability issues are not present.
 - Simple and well-defined stratigraphy (typically one or two distinct Design Sections).
 - No rock instability issues observed in adjacent exposures (cuts or outcrops).
 - No known or suspected adverse discontinuity conditions; rock is typically massive, hard, and well cemented.
 - Rock mass has not undergone significant deformation due to tectonic processes

- No known or suspected adverse groundwater conditions.
 - No history of rock slope instability within exposed stratigraphic units.
 - No known voids (i.e., mining, karst, etc.) within or immediately beneath the cut slope that may impact or be impacted by the proposed cut slope.
 - An adequate rock catchment area exists and can be safely maintained.
 - Detailed field discontinuity measurements are not required.
 - Detailed rockfall analyses are not required when a preliminary evaluation indicates favorable discontinuity conditions. Favorable conditions include: the dip direction of the discontinuity is greater than 30 degrees of the dip direction of the slope face; the dip of the discontinuity must be greater than the dip of the slope face and therefore does not “daylight” in the slope face; the dip of the discontinuity is less than the angle of friction along the discontinuity.
 - Cut slope geometry is determined to be in accordance with [Section 8.5.1.1](#)
 - Typically does not require the implementation of Hazard Mitigation Systems as discussed in [Section 8.9](#). A subsurface investigation is required to determine Rock Quality Designation (RQD), material break elevations (i.e. changes in stratigraphy and design section type), overburden depth, and groundwater elevations in support of design and to provide information for a construction bid package.
6. **Detailed Rock Cut Slope Design** – A proposed rock cut slope requiring moderate to extensive design effort based on an evaluation of the local geological conditions and extent of proposed construction. Detailed Rock Cut Slope Designs require quantitative analysis and understanding of the structural geology to adequately evaluate the rockfall hazards and the risk to the traveling public, service interruption, and Department facilities. Existing geologic and subsurface data should be evaluated and used to plan a subsurface investigation. The subsurface investigation should be designed to obtain data and information that is absent but required for design. All rock cut slopes not considered a Simple Rock Cut Slope Design should be considered a Detailed Rock Cut Slope Design. The following general conditions are characteristics representative of Detailed Rock Cut Slope Designs:
- Potential high risk to traveling public, Department facilities, and/or service interruption.
 - Proposed rock cut has a height greater than or equal to 20 feet.
 - Predominant discontinuity is oriented at an angle greater than 20 degrees out of the proposed cut slope face and at an angle greater than the assumed friction angle along the discontinuity.

- The general dip direction of the predominant discontinuities falls within 30 degrees of the dip direction of the existing or proposed cut slope face (refer to general structural geology terms for explanation of terms).

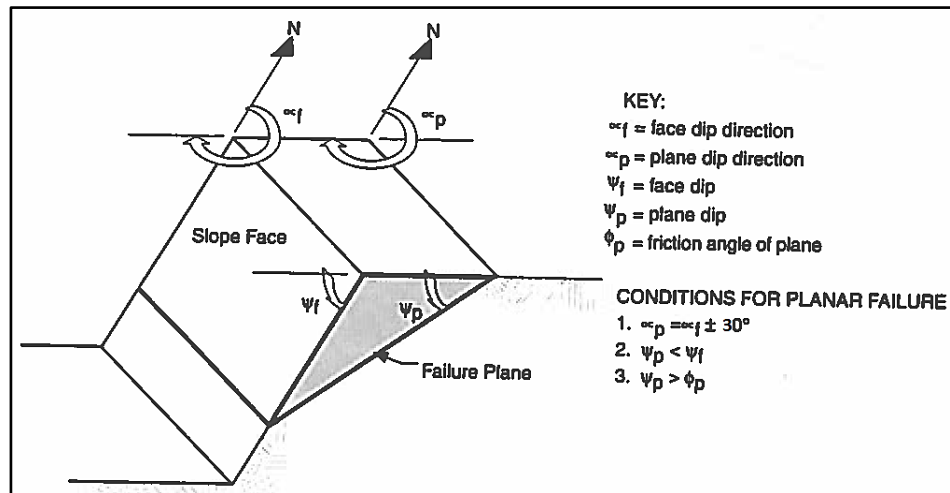


Figure 8.2-5 – Diagram Detailing Conditions for Planar Failure

- Preliminary kinematic assessment(s) indicates moderate to high potential for global instability.
- Structural features such as folds, faults, and shear zones or discontinuity orientations that could contribute to global instability, are present within the rock mass.
- Adjacent exposures (cuts or outcrops) exhibiting evidence of rock instability.
- Evidence of or potential for adverse discontinuity patterns.
- Evidence of or potential for adverse groundwater conditions.
- Exposed lithologic units associated with known record(s) of rock instability or known to be highly susceptible to deterioration.
- Presence of voids (i.e. mining, karst, etc.) within or immediately beneath the cut slope that may impact or be impacted by the proposed cut slope.
- Cut slope geometry is determined in accordance with [Section 8.5.1.1](#) if located in Region I or [Section 8.5.1.2](#) if located in Region II
- Often requires the implementation of Hazard Mitigation Systems (Refer to [Section 8.9](#)).
- Requires detailed investigation of geologic structure, which includes field collection of rock mass discontinuity data in support of kinematic and rockfall model evaluations.
- Subsurface investigation is required to determine Rock Quality Designation (RQD), material break elevations (i.e. changes in stratigraphy, zones of weathering, overburden depth), groundwater conditions, and discontinuity orientations for support during design.

8.3 DESIGN SCOPE OF WORK

Each existing or proposed rock cut slope evaluation presents a unique set of variables to consider and are dependent upon project requirements, the surface/subsurface conditions, and proposed construction. The level of effort required to investigate, design, and/or develop mitigation measures for each slope depends upon several variables including the height of the slope, site-specific geology, currently available information, existing test boring information, available right-of-way, acceptable risks, project objectives, and other site-specific factors. Therefore, each individual rock cut slope must be thoroughly examined **to identify any rockfall that could cause injury, damage, or service interruption**. Proper identification requires a combination of a thorough understanding of the geologic conditions (rock types, quality, and geologic structure/discontinuities), slope geometric conditions, and traffic conditions. This section describes the Department's approach to preparing the scope for design while considering these variables. Sources of information currently available include previous geologic reports and studies, existing geotechnical engineering reports, existing test borings, and exposed rock cuts or natural outcrops in the immediate area.

Department projects consist of new roadway sections, improvements (e.g., widening or interchange realignment requiring numerous new cut slopes in bedrock of varying proportions), and bridge replacement projects. Alternatively, the sole purpose of a project may be to perform full scale rehabilitation and improvement of an existing rock cut in order to decrease the risk to Department facilities and public safety. Similar to pavements and structures, rock cut slopes have a design life, and perform satisfactorily for a period of time. Through weathering and gravity, a rock cut slope will at some point reach a state of equilibrium. Evaluations and design of any proposed rock cut or slope rehabilitation should be discussed with the DGE at the beginning of each project. If the project involves the mitigation of an existing rock cut slope the DGE may have existing data and information available concerning the materials, structure, and/or geometry of the slope and roadway, and/or possible maintenance records of rockfall history, cleanup, and slope maintenance or repair to assist in development of a conceptual design. The review of available information should include published geologic information and existing project records (i.e., subsurface data, design plans, and as-built information, if applicable).

Field reconnaissance should be conducted to verify and supplement existing information. Refer to Sections [8.4.1](#) and [8.4.2](#) of this Chapter for reviewing preliminary information obtained during office reconnaissance and for conducting field reconnaissance for rock slope and catchment design. Refer to [Chapter 2](#) of this publication for additional guidance when performing office reconnaissance and conducting field reconnaissance.

During the office and field reconnaissance, the results should cover a basic understanding of the complexity of the proposed rock cut slope, identify the design tasks necessary to address the existing and proposed conditions, and provide likely design solutions based on criteria and guidance provided in [Section 8.5](#). Additionally, assumptions pertaining to incomplete or uncertain information must be identified at this stage. Information obtained during later phases of investigation that deviate from initial project assumptions may require revisions to the scope to perform additional investigations and/or evaluations. The intent of the office and field

reconnaissance is to verify project conditions and assumptions and reduce inconsistencies that may develop in the scope during the project execution phase.

The complexity of rock cut slope designs may be classified as Simple or Detailed based on the results of the office and field reconnaissance.

Once the slope has been classified as either a Simple or Detailed Rock Cut Slope Design, a project-specific scope of work through the design phase is developed. Additional guidance regarding the development of the scope of work for rock cut slope designs is, as follows:

- The rock cut slope design classification system is intended as a guide and starting point to develop the scope of work. Engineering judgment must be used during the scope development process. Each scope of work for rock cut slope design will be reviewed and approved by the DGE. Scopes of work should specifically identify the anticipated rock cut slope design classification and describe the specific work tasks envisioned from the investigation phase through project completion.
- Rock cut slope design for a new slope requires a balance between minimizing the construction/right-of-way acquisition costs, the need for future maintenance, and protecting the Department's facilities and public. Slope angles between near vertical, to 1:1 are commonly designed. However, in some geological conditions, flatter slope angles between 1:1 and 2:1 are allowable or necessary to achieve project demands. Preliminary cut slope designs should enable construction of the cut slope without the need for additional right-of-way and/or implementing a Rockfall Hazard Mitigation System. In the event this is not possible, an alternative analysis may need to be performed to discover the most effective way to mitigate the amount of right-of-way necessary by implementing a Rockfall Hazard Mitigation System (e.g., rock bolts, shotcrete, slope mesh, rockfall barrier fence, etc.). Any assumptions that were made to fulfill the alternatives analysis part of the preliminary design should be documented within the project scope.
- The design process for rehabilitation of existing slopes follows the same approach as that for new cut slopes. Slope rehabilitation projects will typically be classified as a Detailed Design due to the presence or potential of rockfall. Often, parameters for quantitative rockfall analysis modeling can be obtained during field reconnaissance (e.g., measurements obtained from the existing slope face, size of fallen debris/rocks, etc.), and without the need for subsurface investigation. Design solutions for slope rehabilitation projects are more likely to require rockfall control and mitigation systems when compared to new rock cut slope designs. This is discussed more in [Section 8.9](#).
- For Detailed Rock Cut Slope Designs, at least one rockfall analysis model should be generated and evaluated at a critical section. The critical section is usually at the highest proposed slope elevation, but may vary based on the local geological conditions or other factors (e.g., slope ratio, proximity of facilities to the toe of the slope, etc.). Where the proposed slope conditions vary such that the critical section is not clear, multiple rockfall analyses are warranted. Long rock cut slopes of consistent height should be scrutinized closely in order to identify

critical sections due to anticipated geological conditions and/or cut slope geometries.

- The greater the complexity and level of effort required to complete the rock cut slope design or mitigation design the greater the effort required to complete the geotechnical investigation.

8.4 GEOTECHNICAL/GEOLOGICAL INVESTIGATIONS

Geotechnical investigation activities for the design and construction of a rock cut slope include the following: office and field reconnaissance, detailed geological structure mapping, subsurface exploration, and remote sensing.

Office and field reconnaissance must be conducted as part of the investigation for all rock slope work. Subsurface exploration will always be required for new rock slopes, but may or may not be required for the rehabilitation of existing slopes depending upon the degree to which the existing lithology and geologic structure can be determined from the exposed cut face. Remote sensing techniques will typically be most applicable for existing rock cuts. The need and practicality for remote sensing will depend upon the complexity of the geologic structure, site geometric conditions, and site accessibility.

8.4.1 Office Reconnaissance

The office reconnaissance should be performed in accordance with [Chapter 2, Section 2.2](#) of this publication, prior to completion of the field reconnaissance. The findings of the office reconnaissance should uncover all available geological and geotechnical information applicable to a rock cut slope design or mitigation project. This data is then verified, or further clarified, during the field reconnaissance. The office reconnaissance for rock cut slopes should specifically include, but is not limited to, the review of the following records:

- Geological information (maps and reports)
- Previous geotechnical engineering reports
- Aerial and oblique photographs and orthoimages
- LiDAR topography
- Existing Department maintenance records (e.g., frequency of rockfalls, seasonality, and size of rock blocks removed)
- Pennsylvania Geologic Survey's (2005) map of geologic units containing potentially significant acid-producing sulfide minerals (Note: The Designer is referred to [Chapter 10](#) of this publication if acid producing rock is encountered on a Department project).
- PADEP Mining Records

For a more extensive and detailed list of geological and geotechnical sources of information to assist in the design of rock cut slopes or mitigation refer to [Chapter 2, Section 2.2](#) of this publication. Completion of the office reconnaissance creates a foundation for the geotechnical issues or concerns related to the design or mitigation of the rock cut slope.

8.4.2 Field Reconnaissance

A field reconnaissance will be conducted after the completion of the office reconnaissance. The field reconnaissance should focus on collection of missing geological data and augmenting or verifying existing data with site conditions. The main objective of the field reconnaissance is to obtain or verify that the geotechnical information can be used to complete a geological model. This model will form the basis to determine rockfall potential. The field reconnaissance should be performed in accordance with [Chapter 2, Section 2.3](#) of this publication.

If the field reconnaissance indicates additional geological information is needed to proceed with design, the data should be acquired while onsite. Depending upon existing information and site specific requirements, a subsurface investigation or remote sensing investigation may be warranted.

If an existing rock cut slope, or a rock slope in proximity to the proposed slope, contains the same design units and geologic structure, the following questions should be addressed, as follows:

- Is there currently a rockfall problem? If so, where is the rockfall problem located?
- What is the nature and frequency of past rockfall events at these sites?
- Where are the rockfall source area(s), travel path(s), and runout zone(s)
- What is the slope geometry (e.g., inclination, height, roughness, etc.) and slope material properties?
- What is the likely motion of the rock traveling down the slope (rolling, bouncing, or sliding)?
- What size rocks have reached the base of the slope versus their size in the source zone?
- Does fallen rock at the base of the slope appear to have broken apart during descent down slope?
- How far do the rocks roll past the base of the slope (or point of interest)?
- What type of rock mass(es) are present while considering lithology, durability, and hardness?
- What is the groundwater or seepage conditions?
- What are the conditions of the discontinuities present within the rock mass (e.g., roughness, persistence, infilling, orientation)?
- Are vegetation or tree roots contributing to slope instability?

8.4.2.1 Detailed Geological Structure Mapping

If the project, requires a Detailed Rock Cut Slope Design, detailed geological structure mapping should be completed using either a scanline mapping or by window mapping. However, depending upon the complexity of the project and the need for geologic structural data, remote sensing technology is recommended by the Department. This technology, namely LiDAR and terrestrial digital photogrammetry, are discussed in further detail in [Section 8.4.2.3](#)

Scanline mapping is performed by stretching a tape approximately 150 to 300 feet across the rock exposure. The distance along the line is noted for each discontinuity intersected by the tape and the properties of each discontinuity are noted. This mapping technique is better suited for larger bedrock exposures, primarily existing rock cuts. When the orientation (trend and plunge) of the scanline is known, the true spacing of the joints belonging to the various joint sets encountered can be calculated from their apparent spacing along the scanline. The frequency with which discontinuities of a particular set will intersect the scanline is dependent on the strike of the set. In the case of a joint set striking parallel to the scan line, no discontinuities may intersect the scanline. The frequency of intersection may also depend on the length of joints. Depending on spacing, a set of joints having relatively short lengths may intersect the scanline less frequently than a set of joints having greater lengths. Corrections for observation bias have been developed for scanline sampling (Terzaghi, 1965). Performing at least two scan lines oriented perpendicular to one another will reduce observational bias.

Conversely, window mapping consists of mapping all discontinuities within a “window,” typically 100 ft² in area, on a rock exposure and recording the properties of each discontinuity. This mapping technique is typically better suited for smaller isolated bedrock exposures. The sampling window may be delineated by forming a rectangle of measuring tapes fastened to the rock face. The window should be as large as possible to minimize sampling bias. Window sampling reduces the observation bias due to discontinuity orientation and length associated with scanline sampling; however, bias may remain because of curtailment of the discontinuities due to outcrop cover. Ideally, at least two windows of a similar size on adjacent rock exposures that have different (preferably orthogonal) orientations should be mapped. This will avoid bias with respect to discontinuities that are nearly parallel to the rock face.

Obtaining geological discontinuity data from two perpendicular rock exposures is very challenging and often not available. If during the reconnaissance phase it is determined that discontinuities parallel to the slope face are of high concern regarding global slope stability, an additional investigation to assess the slope parallel discontinuities should be conducted. This can be accomplished by either exposing the rock mass in plan view, excavating test trenches, or drilling angled core borings and surveying any intersected discontinuities with a downhole optical televiewer.

8.4.2.2 Subsurface Exploration

For any proposed rock cut slope (Simple or Detailed design), a subsurface exploration program will be required by the Department to obtain the necessary data for design and construction. For existing rock cut slopes, the need to complete a subsurface investigation will be determined on a case-by-case basis depending upon the amount and quality of existing geotechnical data available, and if additional data can be collected for the exposed rock face in lieu of a subsurface investigation. Also, the Designer must consider drill rig accessibility constraints on an existing rock cut slope. The need to complete a subsurface exploration program for existing rock cut slopes should be determined during the field reconnaissance where the Designer reviews with the DGE the existing information and design objectives.

Subsurface explorations should be performed in accordance with [Chapter 3](#) of this publication and Publication 222. The subsurface investigation may include angled boreholes and downhole video/acoustic televueing to determine discontinuity orientations and conditions. The number of borings required is determined from the site specific geologic conditions. The presence of highly folded and faulted stratigraphy, mine voids, and karst features, would likely require more borings than a site underlain by a competent design unit with occasional random joints.

8.4.2.3 Remote Sensing

Remote sensing technologies such as digital terrestrial photogrammetry and ground-based LiDAR systems are excellent tools for the Designer. This technology can collect a large quantity of information about the surface topography and discontinuities present in an existing rock slope or natural bedrock exposure. It can also be used to monitor rock slope movement and allows data collection from portions of a rock cut slope that are largely inaccessible. The Department recommends the use of remote sensing technology for the collection of geologic discontinuity data. Both types of remote sensing techniques are discussed in more detail in the following paragraphs.

Digital terrestrial photogrammetry works on the principle that sight rays from an object will appear at different locations on a camera's image sensor as the camera location changes. By a process of triangulation, pairs of image points recorded on the camera's image sensor in overlapping (stereo) pairs of photographs are used to determine an object's location. Photogrammetry software can quickly process digital images and calculate the 3-D coordinates of thousands of points in overlapping images and provide a high resolution 3-D image model of a rock cut slope face or rock exposure. The orientation of discontinuities can be measured using the digital terrain model (DTM) developed from the 3-D image model (Haneberg et al, 2006; Haneberg, 2008; Tonen and Kottenstette, 2007; and Roman and Johnson, 2012). Digital terrestrial photogrammetry provides photographic documentation of the rock slope being investigated. The 3-D model developed is orthorectified; therefore, the rock slope image can be projected onto a vertical reference which can be used for engineering design.

Ground-based LiDAR scanners can also be used for outcrop mapping (Kemeny and Turner, 2008; Kemeny et al, 2011; Watts, 2011). LiDAR scanners determine distance based on the time of travel of an emitted and reflected laser pulse. The time of travel is either measured directly or by measuring the phase shift of a sinusoidal modulated laser pulse. Some scanners can measure more than one arrival time for an emitted pulse, which provides a means to filter out vegetation. Pulses are emitted and received in quick succession (thousands of times per second), creating millions of distance measurements along with a measurement of the intensity of the returned pulse. The data, which are referred to as "point clouds," are used to render a shaded image of the surface being mapped. The image may be colored by associating color values from a digital image with the location of each point to produce a near-photographic quality image.

In-depth details of conducting remote sensing techniques for mapping rock cut slopes are beyond the scope of this publication. It is recommended that the external technical references cited in this section be reviewed to understand remote sensing technologies, capabilities, and limitations, and assess their applicability for a specific project.

Remote sensing technologies may be useful or in some cases necessary in the evaluation and mitigation of existing rock cut slopes and natural bedrock exposures. Access limitations due to slope height, slope angle, roadway geometry, traffic volume, active rockfall or highly unstable slope conditions often justify the use of remote sensing techniques to collect geologic discontinuity data. This technology reduces human risk associated with traffic and active rockfall when collecting geological discontinuity data manually. When remote sensing technologies are employed during field reconnaissance, it can minimize the risk during the design and construction phases of the project. Also, remote sensing techniques may be especially valuable when complex geologic structures are present, a large population or rock mass discontinuity data is required for stereographic analysis, and/or where global stability is a high concern.

8.5 ROCK CUT SLOPE DESIGN FOR STABILITY

The primary objective of a rock cut slope design is to construct a rock cut that will be safe during construction and will provide long-term safety to Department facilities and the public in the future. Due to the nature of rock masses and the geologic settings in which they are found, nearly every rock slope design is unique.

The construction cost of a rock cut slope and the impact to private property are issues that must be balanced with rock slope stability design. In most cases a rock mass will contain more than one set of discontinuities which intersect the proposed or existing rock cut face. With each degree of slope angle reduction, fewer and fewer intersections are exposed in the rock cut face. This in turn, increases the overall stability of the rock cut. With each degree in reduction of the slope angle, more rock excavation will be required and there will be adversely an associated increase in cost.

In general, the Designer should target a design factor of safety of 1.5 by completing a limit equilibrium analysis; however, based on engineering judgment, values outside of this range may be appropriate, depending on the site-specific conditions. The factor of safety to be used in stability analyses for a specific rock slope depends on factors such as:

1. The degree of uncertainty in the stability analysis inputs, specifically the amount of intact rock, rock mass strength, discontinuity spacing, discontinuity shear strength, and groundwater conditions
2. The level of investigation and data collection
3. Costs, acceptable risks to the travelling public, and future maintenance costs
4. If the slope is temporary or permanent.

8.5.1 Design Guidelines based on Regions

As previously discussed in [Section 8.1.3](#), separate design guidelines have been established based on geologic and physiographic location. The design guidelines are discussed in detail in the subsequent sections. The many different rock types found throughout the Commonwealth and their engineering properties developed the basis for design of rock cut slopes and is presented in [Table 8.5.1-1](#).

Table 8.5.1-1 – Basis for Design by Region

Physiographic Region	Region I	Region II
Provinces	Appalachian Plateaus and Central Lowlands Provinces	Ridge and Valley, Piedmont, and New England Provinces
PennDOT Districts	<ul style="list-style-type: none"> Districts 1, 10, 11, and 12 Northern and western portions of District 2 Northern portions of Districts 3, 4, and 5 Western portion of District 9 	<ul style="list-style-type: none"> Districts 6 and 8 Southeastern portion of District 2 Southern portions of Districts 3, 4, and 5 Eastern portion of District 9
Basis for Design	<ul style="list-style-type: none"> Use Rock Quality Designation (RQD) and slake durability data Perform kinematic analyses as indicated by field conditions 	<ul style="list-style-type: none"> Perform kinematic analysis to evaluate global stability Perform limit equilibrium analyses as indicated by kinematic analyses and field conditions Consider RQD and slake durability data

After determination of the region in which the existing or proposed rock slope is located, [Figure 8.5.1-1](#) provides guidance for developing and executing the basis of design for rock slope design. Further design guidelines for rock slope projects located in Regions I and II are presented in Sections [8.5.1.1](#) and [8.5.1.2](#) respectively.

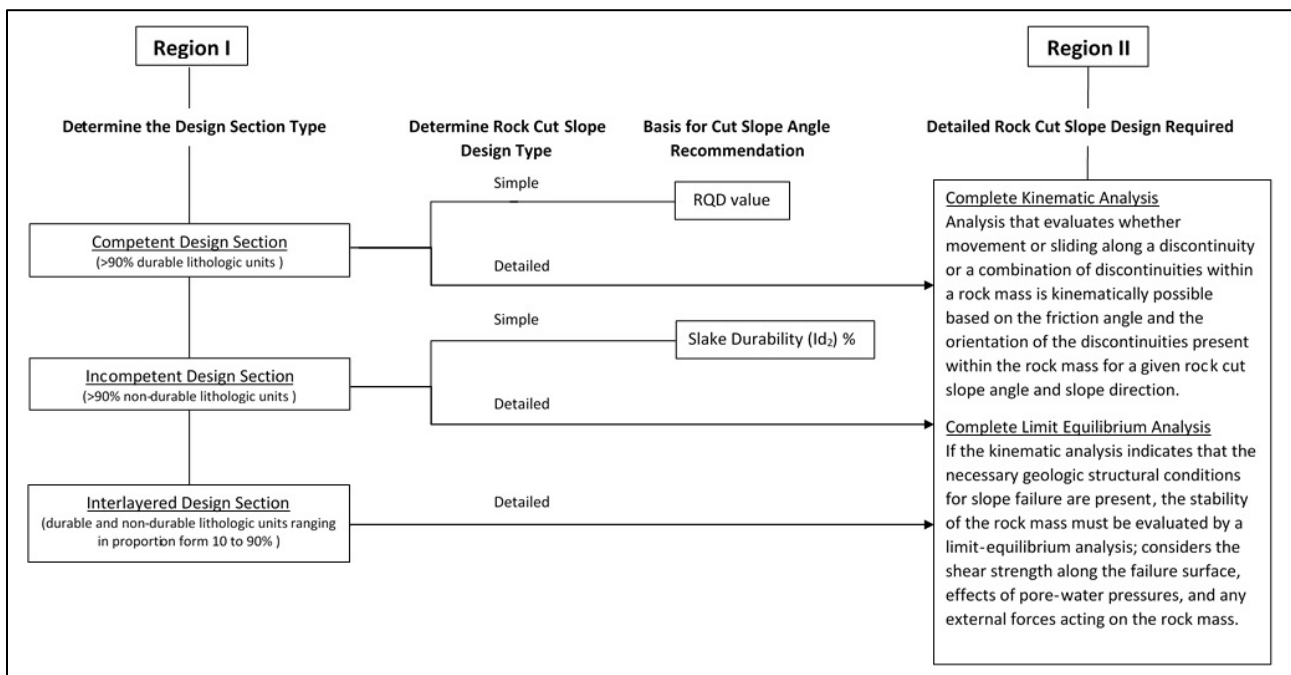


Figure 8.5.1-1 – Rock Slope Design Summary Flow Chart¹

Notes: 1. Determine Region based on [Figure 8.1.3-1](#)

8.5.1.1 Design Guidelines for Projects in Region I

The guidelines for design of rock cut slopes in Region I are based on the presence of flat to slightly dipping, cyclical layers of sedimentary rock having varying degrees of strength and durability and featuring vertical to near vertical orthogonal jointing. The sequence and proportions of the competent and incompetent rock units directly impact the susceptibility and degree of slope undercutting. The spacing, continuity, and orientation of the discontinuities within the competent rock layers define the size of potential rock masses.

The following recommendations should be consulted for rock cut slope design within Region I. The guidelines detail general rock cut slope design recommendations for durable, non-durable, and interlayered design units as described in [Section 8.2](#). The guidelines do not conform to all rock slope conditions; therefore, the Designer must use good engineering judgment and evaluate their applicability to each rock cut slope project. It should be noted that the design procedures have been largely developed by the Ohio Department of Transportation (ODOT); however, some modifications have been made at the discretion of the Department. These procedures largely adopted from ODOT are reasonable as the geologic structure and interbedded sequences of durable and non-durable bedrock units are similar to those found in Region I.

The design of rock cut slopes is a staged process. The number of steps depends on the size and complexity of the project. A Detailed Rock Cut Slope Design may require several phases of investigation, evaluation and design. Conversely, one phase of investigation, evaluation, and design will normally be sufficient for a Simple Rock Cut Slope Design.

The rock cut slope configuration is influenced by several factors (e.g., lithology, rock mass strength, weathering resistance, discontinuity characteristics, etc.). Three rock slope design sections (i.e., competent, incompetent, and interlayered), as defined in [Section 8.2](#), represent the general lithologic conditions typically encountered in rock cut slopes in Region I.

The following subsections outline guidelines for each of the Design Section types within Region I. The following guidelines do not apply to rock cut slope designs which have been classified as a Detailed Rock Cut Slope Design based upon the general conditions as defined in [Section 8.2](#), unless the design section is interlayered. In these situations, additional rockfall analyses may be required to support the proposed cut slope design as discussed in [Section 8.5.1.2](#).

8.5.1.1.1 Competent Design Sections

A competent design section will have a rock slope comprised of one or more durable lithologic units. The cut slope inclination may then be determined based on the Rock Quality Designation (RQD; Deere and Deere, 1998) in accordance with [Table 8.5.1.1.1-1](#).

Table 8.5.1.1.1-1 – Cut Slope Angle Recommendations for Competent Design Sections¹

RQD (%)	Recommendations
<50	Cut Slope at maximum 1H:1V Slope Ratio.
50-75	Cut Slope between 1H:1V to 0.5H:1V Slope Ratio. Evaluate discontinuity orientations with respect to range of cut slope ratios.
>75	Cut Slope to 0.5H:1V, or 0.25H:1V for very thickly bedded units.

Notes: 1. Reference: ODOT, Rock Slope Design Guide (2011), (modified)

For Competent Design Sections, consider other geologic conditions which may impact the performance of the slope in the long term, (e.g., presence of thin clay seams, incompetent and highly weathered rock intervals, beds susceptible to loss of strength over time, seepage conditions, discontinuity orientation, discontinuity roughness, etc.) regardless of the RQD for the Design Section. Design units with closer spacing of discontinuities are prone to higher frequencies of rockfalls and potential global stability issues. Apply sound engineering judgment based on the results of the geologic characterization to determine the proposed cut slope angle. Consultation with the DGE is recommended.

8.5.1.1.2 Incompetent Design Sections

An incompetent design section will have a rock slope comprised of one or more non-durable lithologic units. In addition, the rock slope will meet the criteria of a Simple Rock Cut Slope Design. The cut slope inclination may be determined based on the average second-cycle Slake Durability Index, SDI (Id_2) as shown in [Table 8.5.1.1.2-1](#). The SDI (Id_2) is obtained by completing the Slake Durability Test according to ASTM 4644 where dried rock fragments of a known weight are placed in a drum fabricated with 0.08-inch square mesh wire cloth. The drum is rotated and partially submerged in distilled water. The specimens remaining in the drum are dried at the end of the rotation cycle (10 min at 20 rpm). After two cycles, the dry weights of the specimens are recorded and the SDI (Id_2) is calculated (weight retained/initial weight) x 100. The test simulates the natural degradation of clay bearing rocks when subjected to alternating cycles of wetting and drying. Slaking is a degradation process whereby clay bearing rocks such as shales, and claystones undergo mechanical breakdown due to absorption and retention of water.

Table 8.5.1.1.2-1 – Cut Slope Angle Recommendations for Incompetent Design Sections¹

SDI (Id_2) (%)	Recommendations
$Id_2 < 20$	2H:1V or flatter
$20 \leq Id_2 < 60$	2H:1V to 1.5H:1V
$60 \leq Id_2 < 85$	1.5H:1V to 1H:1V
$85 \leq Id_2 \leq 100$	1H:1V or steeper

Notes: 1. Reference: ODOT, Rock Slope Design Guide (2011), (modified)

A Slake Durability Testing Program which identifies the number and location of test samples should be established and approved by the DGE prior to testing. At a minimum, two

Slake Durability Tests should be completed when the project is a Simple Rock Cut Slope Design comprised of only one non-durable lithologic unit. If multiple non-durable lithologic units comprise an Incompetent Design Section, additional testing of these units is recommended. An average SDI value is to be used and applied to each design section.

Additional evaluations and/or considerations may be required where the SDI (Id_2) is less than 20 percent. For example, Pittsburgh red beds, primarily comprised of claystone, can have a residual angle of internal friction as low as 12 to 15 degrees and therefore a 3:1 of flatter slope may be required. Consequently, the recommendations listed in the [Table 8.5.1.1.2-1](#) provide an ideal starting point for the cut slope angle while still considering the evaluation of other site specific geotechnical engineering properties of the soil and rock. Given the likelihood that the non-durable lithologic units will weather and deteriorate over time, it may be warranted to model the rock as an equivalent soil mass and perform global stability analyses to determine cut slope stability. Also, consider if poor rock core recovery is obtained during the drilling program at a sample interval intended for Slake Durability Tests. In this instance, the Designer could conclude this lithologic unit is likely susceptible to slaking resulting in low durability. Slake durability is discussed in more detail in [Chapter 15](#) of this publication.

Special care should be taken to recognize and identify the location, condition, and orientation of discontinuities that may control the overall rock mass strength, and control the long-term stability of the cut slope. In these cases, the proposed slope should be considered a Detailed Rock Cut Slope Design. Design and slope stability rockfall evaluations, along with an additional kinematic analysis, should be performed.

8.5.1.1.3 Interlayered Design Sections

Interlayered design sections consist of durable and non-durable lithologic units each ranging in proportion from 10% to 90%. These units exhibit significant stratigraphic variability and must be considered as a Detailed Rock Cut Slope Design. The cut slope design recommendations set forth in this section may be applicable for these design sections individually; however, in order to account for stratigraphic variations, multiple slope ratios and geotechnical (lithologic) benches must be considered to allow for differential weathering of the interlayered units.

8.5.1.2 Design Guidelines for Projects in Region II

The guidelines for the design of rock cut slopes located in Region II differ significantly from Region I guidelines due to the following geological conditions:

- Generally, the most persistent discontinuity within the rock mass is bedding or foliation. This may be variable as beds can be steeply inclined or even overturned or part of a thrust sheet due to faulting. Unlike Region I, bedding or foliation can be oriented at an angle steeper than the coefficient of friction along the bedding or foliation discontinuity in Region II. In Region II, faults and shear zones may be within the project area and play a major role during the stability analysis and design or remedial design. In addition, tectonic joints are pervasive in Region II

with highly variable and irregular patterns. Due to the aforementioned conditions, at a minimum, a global stability analysis is required.

- The rocks within Region II are generally more durable than rocks located in Region I and less susceptible to slaking ($I_d > 85\%$). Claystones are generally not present in appreciable amounts within Region II, and the shales are typically more indurated and characterized by a higher degree of cementation than the shales located in Region I.
- Due to complex folding and faulting, the rocks within Region II may exhibit a wide range of discontinuity orientations within a small geographic area. Thus, one rock cut slope may exhibit several rockfall failure modes. For example, in the case of a rock cut for a north-south trending highway, toppling may be the predominant failure in the east-facing cut slope, while planar failure may predominate in the west-facing cut slope.
- The principle failure modes within Region II are planar sliding, wedge sliding, toppling, and rockfall induced by differential weathering.

When considering these geological conditions described above, the design of rock cut slopes in Region II, must consist of two phases. The first phase consists of a global stability evaluation, and the second phase consists of a rockfall hazard evaluation. Therefore, proposed rock cut slope designs should be considered Detailed Designs as defined in [Section 8.2](#). **Due to global stability concerns, Tables [8.5.1.1.1-1](#) and [8.5.1.1.2-1](#) should not be consulted for recommended cut slope angles.** A kinematic analysis is required to evaluate the rock mass for global stability analysis and is summarized briefly in the following section.

8.5.2 Kinematic Analysis Method

The kinematic analysis evaluates whether movement or sliding along a discontinuity or a combination of discontinuities within a rock mass is kinematically possible based on the friction angle and the orientation of the discontinuities present within the rock mass for a given rock cut slope angle and slope direction. In other words, the analysis determines if the orientations (dip and dip direction) of the various discontinuities will interact with the cut slope orientation and inclination to form discrete blocks with the potential to fail. This analysis is accomplished through a stereographic analysis and is often referred to as a kinematic analysis even though it is a static analysis. The analysis is purely geometrical and does not consider the magnitude of the forces acting on the rock mass. A complete discussion of kinematic analysis is beyond the scope of this publication; however, a brief summary outlining major aspects of the analysis is provided in the following subsections. The Designer is encouraged to consult *Rock Slope Engineering* by Hoek and Bray (1981), which has been incorporated in the Federal Highway Administration's *Rock Slopes Reference Manual* (Munfakh et al., 1998) for additional guidance.

8.5.2.1 Field Discontinuity Data Collection

It is essential to collect a representative population of discontinuity measurements from the rock mass under investigation so the geometrical characteristics of the rock mass discontinuities can be satisfactorily determined. The number of discontinuity data points needed to fully characterize the rock mass can vary from project to project based on its complexity and

the potential hazard (e.g., injuries, property and roadway damage, etc.) if a rockfall event were to occur. In addition, groundwater conditions and the strength of the rock mass play a role in determining a representative discontinuity data set. The purpose of defining the geometrical characteristics of the discontinuities within the rock mass is to provide a basis for selecting the most appropriate failure mode. This is one of the most important decisions in the entire process of rock slope stability investigation and analysis.

Methods for measuring discontinuities are discussed in [Section 8.4.2.1](#). In the case of designing new rock cut slopes where no existing outcrops are available, discontinuity measurements may be obtained from rock core samples collected from test borings, or, less reliably, from nearby outcrops within the same lithological and structural setting. Literature (maps and reports) published by the Pennsylvania Geological Survey or the United States Geological Survey may provide some information regarding structural orientations. Please reference [Chapter 2](#) of this publication for a more comprehensive list of available published literature and websites.

8.5.2.2 Graphical Presentation of Discontinuity Data

The rock mass discontinuity data are plotted on a spherical projection so that they can be evaluated and incorporated into kinematic analyses. The stereographic projection is then accomplished by plotting the data on an equal area stereonet often referred to as a Schmidt stereonet. The structural discontinuity data are typically plotted in the pole format in order to detect the presence of preferred orientations, which define discontinuity sets and determine mean and extreme values of the orientations of the discontinuity sets. This process is typically facilitated by contouring the poles to distinguish the repetitive discontinuities from random or unique discontinuities. Planes representing the discontinuity measurements may also be plotted and contoured as dip vectors. Plotting and contouring discontinuity data may be done manually; however, it is tedious, time-consuming, and prone to errors. The Designer is encouraged to use one of the many commercially available computer software programs indicated in [Section 8.5.2.3](#) that plot and contour rock mass discontinuity data.

8.5.2.3 Kinematic Analysis of Discontinuity Data

The contoured discontinuity data are used to kinematically evaluate the potential for planar sliding, wedge sliding, and toppling modes of failure. This is accomplished by evaluating the necessary structural conditions required for each of the listed modes of failure. Computer software programs (e.g., RockPack III (Watts, 2003), Dips (Rocscience, 2012), DipAnalyst (Admassu, 2012)) may facilitate performance of the kinematic analysis. The kinematic analysis should be based on rock slope evaluation procedures developed by Hoek and Bray (1981), which have been incorporated in the Federal Highway Administration's Rock Slopes Reference Manual (Munfakh et al., 1998) and the Transportation Research Board's Report on Rockfall Characterization and Control (Turner and Schuster, 2012).

8.5.3 Limit Equilibrium Analysis Method

If the kinematic analysis indicates that the necessary geologic structural conditions for slope failure are present; the stability of the rock mass must be evaluated by a limit-equilibrium analysis, which considers the shear strength along the failure surface, effects of pore-water pressures, and any external forces acting on the rock mass. A detailed explanation of the limit equilibrium analysis is beyond the scope of this publication. The Designer is referred to the references in the preceding paragraph. Limit equilibrium calculations, and design of artificial support systems, such as rock bolting, can be analyzed using commercially available computer software.

8.6 ROCK SLOPE BENCHES

A bench is a flat surface typically constructed at prescribed intervals within the rock cut. Benches should be employed with care and designed based on the geologic conditions present within the proposed cut. Benches constructed in certain geologic conditions (e.g., where it is slake susceptible, where less durable rock underlies a more durable rock, etc.) are advantageous, compared to other geologic conditions where benches can actually increase rockfall hazards by providing launching points for rockfall. The following sections provide a detailed discussion of the purpose of benches, which geological conditions are appropriate for benching, and design recommendations.

The purposes of a bench may include:

- Allowing for erosion of a less durable lithologic unit(s) underlying a more durable lithologic unit(s) where weathering may result in undercutting of the overlying unit(s) resulting in rockfall
- Allowing for overall steeper angles of a slope where interbedded weaker lithology's are present
- Providing stages of construction
- Providing transition areas

The Department recognizes three types of benches:

1. Overburden benches
2. Geotechnical (or lithologic) benches
3. Construction (temporary) benches

8.6.1 Overburden Benches

A soil overburden bench limits slope instabilities at the top of a designed rock slope at the transition between exposed rock and overburden soils as depicted in [Figure 8.6.1-1](#). The purpose of the soil overburden bench is to create an area where adjustments can be made during construction due to unexpected variations in the soil-rock interface elevation without requiring a change to cut slope design angles and limits. At the interface between soil overburden and

bedrock, a minimum ten foot wide bench should be provided. An overburden bench is required where the design cut slope is 1H:1V or steeper.



Figure 8.6.1-1 – Example of an Overburden Bench

Slopes in the soil overburden zone should typically be 2H:1V where the slope height is more than ten feet thick. If stability of the proposed overburden is questionable, it may be necessary to perform slope stability analyses to support the proposed design cut slope ratio. A Factor of Safety of 1.5 should be achieved for analyses representing long-term conditions. Special care should be taken to accurately assess the strength parameters, groundwater conditions, and failure mechanisms within landslide-prone glaciolacustrine deposits, colluvial and residual soils, or other soils and highly weathered rock with known stability problems when excavated.

If the overburden zone is less than ten feet thick or the natural slope is 1H:1V or flatter, rounding of the top of the cut slope to blend into the natural slope may be permissible if no other geological conditions are noted that may present a hazard to the overburden slope. If the overburden consists at least partially of highly weathered, soft bedrock, steeper slope ratios may be feasible.

8.6.2 Geotechnical (Lithologic) Benches

A geotechnical (lithologic) bench is a permanent bench constructed at the top of a less durable design section (e.g., shale, claystone, etc.). The aforementioned design section is overlain by a competent design section (e.g., sandstone, siltstone, etc.). Reference [Figure 8.6.2-1](#) for an example of geotechnical bench. The Department recommends that a geotechnical bench only be designed for rock cuts in interbedded strata containing weak, poorly indurated strata and

more durable resistant substrata. The purpose of a geotechnical or lithologic bench is to provide protection against undercutting of the more competent design section as the incompetent design section weathers over time. Most of these sequences of rocks occur in the post Pottsville cyclothem units, which consist of a vertical repetition of sandstone, siltstone, shale, limestone, claystone, and coal. The cyclothem units are located in Districts 10, 11 and 12 and portions of Districts 1, 2, and 9. If a bench is constructed, it is imperative the design include provisions for maintenance and inclusion of access points to all benches as well as a sufficient width for equipment access to account for future weathering.



Figure 8.6.2-1 – Example of a Geotechnical Bench

The Federal Highway Administration does not recommend construction of geotechnical benches in the following geologic conditions:

- non-horizontal (inclination greater than 20°) bedded sedimentary rock,
- competent, high durability sedimentary rock sequences that do not contain highly erodible units (shales, claystone, coal,)
- metamorphic and igneous rocks.

FHWA Publication SA-93-085 (1994) discourages mid-slope benches with the specific intent of catchment because they are rarely cleaned due to inaccessibility and could become launching points for rocks, increasing the rock fall hazard. In general, mid-slope benches are not effective for rockfall control unless they are directly beneath a near vertical slope (0.25H:1V or steeper). The Department does not prohibit the use of geotechnical benches for rock cut slope design; however, detailed kinematic analyses of the rock mass are required, and alternative rock slope designs having no benches must initially be considered. If a geotechnical bench is

proposed, the rock cut slope design must provide a means for cleaning and maintaining the bench and measures to prevent launching of rocks.

The following design guidelines apply to cases where geological conditions necessitate the construction of geotechnical (lithologic) benches (modified from ODOT, 2012):

- For Incompetent Design Sections, 10 feet thick or less, the benches should be 10 feet wide.
- For Incompetent Design Sections thicker than 10 feet, the benches should be made wider to account for the anticipated weathered slope geometry. The Designer should use engineering judgment to determine the site-specific minimum thickness of a Non-Durable Lithologic Unit that will require benching. Conditions to consider include the rate of weathering and the ultimate angle of repose of the weathered incompetent material. This evaluation can be supported by observing slopes in similar stratigraphy, evaluation of the Slake Durability Index (SDI), and assessment of the rock mass characteristics.
- For Interlayered Design Sections, provide a minimum 15-foot-wide bench at the contact between different Design Sections to allow for future access. The Designer should use engineering judgment to determine the site-specific bench size required.
- Where permeable formations overlie impermeable ones (including areas of fractured flow), which may indicate potential aquifer zones, the configuration of benches must consider drainage issues.
- For coal, clay or mineral seams of mineable thickness, or in the case of known or suspected underground mines that will be located within the cut slopes, a fifteen foot wide bench should be designed to allow for drainage and future access. Bench locations should be below suspected mine voids and above unmined seams. More specifically, the bench should not be constructed directly at the top of a coal seam, but rather be constructed in an adjacent overlying lithologic unit, such as a shale, which is commonly located above a coal seam.
- The slope of benches longitudinally should follow the base of the competent rock with the outsloping having positive drainage typically at a grade of 10 percent, with a minimum grade of 3 percent. Special consideration should be given to drainage in vicinity of coal seams.
- Where there are Competent/Incompetent Design Section interfaces near the base of the slope at the catchment ditch, a 5-foot-wide maintenance bench or structural controls should be inserted in the ditch below the road grade so that maintenance of the ditch can be performed without undercutting the toe of the cut slope.
- Where the above guidelines would result in different types of benches in the vicinity of each other (e.g., a construction bench and a geotechnical bench within a few feet vertically), the Designer must use engineering judgment to produce a practical design, which provides precedence for the geotechnical bench.
- Access roads to benches may require additional right-of-way. Sufficient width for equipment access on maintained benches must be considered and evaluated.
- Design bench widths may need to be increased, beyond the typical 10 to 15 feet, in order to maintain a temporary working bench during construction. The

finished grade line of the rock cut slope must consider all vertical relief as well as the working bench widths.

- Geotechnical benching may be field adjusted during construction to correspond with changes in the contact and structure of the stratigraphic unit(s).
- Install a bench drain along the back of benches where groundwater is encountered or anticipated to collect seeping water, and a backslope drain behind the slope crest to reduce runoff on the slope face.

8.6.3 Construction Benches

A construction bench is a narrow bench that is necessary to allow for common presplitting construction practices. [Figure 8.6.3-1](#) provides an example of a construction bench. For design purposes, where design thicknesses or sections of slope designs are greater than 30 feet, blasting must proceed in stages. These benches are provided to account for the required offset between lifts during pre-splitting due to constructability issues as well as for the tool variances that can occur in drilling presplit holes. Without accounting for necessary construction offsets by incorporating construction benches in the slope design, the as-built cut line will either be moved back at the top, impacting project right-of-way, or be made steeper to maintain the plan offset at the toe of cut, which may impact slope stability.



Figure 8.6.3-1 – Example of a Construction Bench

8.7 ROCKFALL CATCHMENT DESIGN

A rockfall catchment area is defined as the area between the roadway edge of pavement and the base of a cut slope, used to restrict rockfalls from encroaching the roadway. Adequate consideration and proper design of rockfall catchment designs will prevent most falling rock from reaching the roadway. The Department views rockfall catchment ditches as the primary rockfall control method along new slopes. Rockfall catchment areas are designed to stop moving rocks before they reach the roadway by dissipating their energy on flat or negatively sloped ground (Turner and Schuster, 2012). The primary features for an effective catchment area are location, width, shape, and substrate.

As discussed in [Section 8.9](#), Hazard Mitigation Systems such as barriers, rock bolts, drapery, or other similar controls are viewed primarily as being solutions to rockfall problems on existing slopes. Hazard Mitigation Systems should be used in the design of new rock cut slopes only in special circumstances (e.g., where available right-of-way to adequately or economically construct the slope, where rockfall catchment ditch may not be feasible, etc.). Where designed, Hazard Mitigation Systems may control rockfall independently or work in combination with benching and rockfall catchment features.

The Department has established a rockfall catchment design criteria in the 95th percentile at the edge of pavement (typically edge of paved shoulder) for any roadway classification. Similarly, FHWA and other State DOT's generally use 95% effectiveness for catchment area design criteria. The most effective method of minimizing the hazard of rockfalls is to control the distance and direction in which they travel. The recommended and most frequently used method to control rockfall in Pennsylvania is to appropriately size the catchment area based on the 95th percentile design criteria.

The Ritchie Ditch Design Chart shown in [Figure 8.7-2](#) has long been used by many transportation agencies to develop dimensions of rockfall catchment areas. One major limitation of the Ritchie ditch design criteria, is that it relies on the use of a deep, flat bottom ditch with a steep, 1.25H:1V foreslope adjacent to the roadway to prevent rocks from rolling up onto the roadway. These deep, steep sloped ditches are typically not used today as they do not meet current AASHTO roadside clear zone safety requirements. A deep, steep-sloped Ritchie Ditch requires some form of guiderail or barrier to keep vehicles from falling into the ditch. As modern roadside clear zones safety regulations were established, several transportation agencies (ODOT and WSDOT) modified Ritchie's design criteria to provide catchment areas having more gentle slopes (3:1, 4:1, 6:1) as an alternate to a deep ditch design. [Figure 8.7-1](#) illustrates a rock cut slope angle effect on rockfall trajectory.

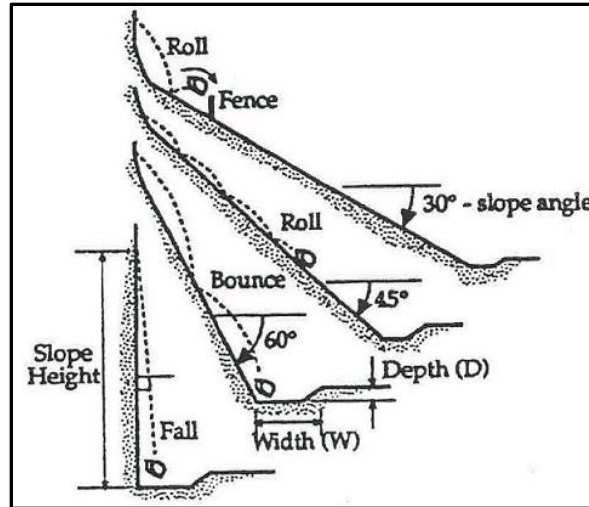


Figure 8.7-1 –Rockfall Trajectory based on Slope Angle¹

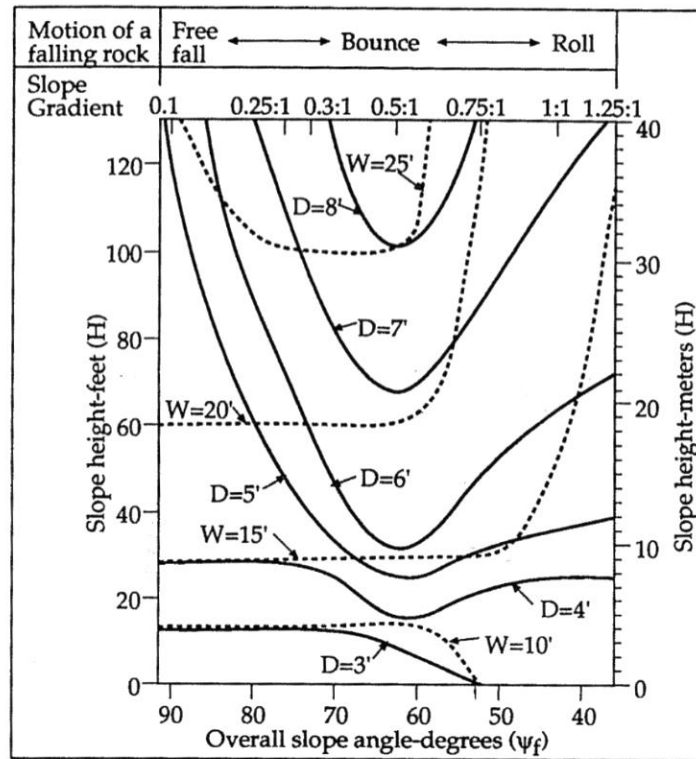


Figure 8.7-2 – Ritchie's Rockfall Catchment Ditch Design Chart¹

Notes: 1. Reference: FHWA (1989)

[Table 8.7-1](#) has been developed based on research conducted by the Federal Highway Administration and other transportation agencies (Pierson et al, 2001). The table provides recommendations for the minimum required ditch dimensions based on rock cut slope height, rock cut slope angle, and catchment ditch foreslope. The depth of the catchment ditch will vary

depending on the catchment ditch foreslope. A steeper foreslope angle will result in a greater ditch depth. For example, a 20-foot wide ditch, with a foreslope angle of 3:1 would result in a ditch depth of 6.7-feet, where as a foreslope angle of 4:1 would result in a ditch depth of 5 feet.

Table 8.7-1 – Recommended Catchment Widths for Varying Slope and Catchment Foreslope Angles^{1, 2}

Catchment Foreslope Angle	Cut Slope Angle	Cut Slope Height, H (ft.)					
		0-40	50	60	70	80	>90
		Catchment Ditch Width, W (ft.)					
3H:1V	0.25H:1V	10	15	15	15	20	25 min.
	0.5H:1V	10	15	20	20	20	25 min.
	1H:1V	15	20	20	20	25	30 min.
	1.5H:1V	15	20	20	20	25	30 max.
4H:1V	0.25H:1V	10	15	20	20	25	30 min.
	0.5H:1V	15	15	20	20	25	30 min.
	1H:1V	15	20	20	25	30	35 min.
	1.5H:1V	15	20	20	25	30	35 max.
6H:1V	0.25H:1V	15	20	25	30	35	40 min.
	0.5H:1V	20	20	25	30	35	40 min.
	1H:1V	25	25	30	35	40	40 min.
	1.5H:1V	25	25	30	35	40	40 max.

- Notes: 1. Reference: ODOT, Rock Slope Design Guide (2011)
2. Values are based on 95% rockfall containment within the catchment area

A catchment ditch for Simple Rock Cut Slope Designs may be sized using the guidance provided in [Table 8.7-1](#) or the Ritchie Ditch Design Chart, [Figure 8.7-2](#). For Detailed Rock Cut Slope Designs, rockfall simulation modeling software should be used to size the catchment area in addition to considerations of [Table 8.7-1](#) and the Richie Ditch Design Chart, [Figure 8.7-2](#). The results of the rockfall simulation model must demonstrate 95% rockfall catchment within the ditch. The catchment area design should be the larger of the catchment dimensions determined by the simulation modeling and the design charts.

In the case of rock cut slopes with multiple slope angles (i.e., presence of benches), the governing slope angle that will be used to determine adequate catchment should be the angle of the portion of the slope that intersects the ditch. Catchment width for an individual rock cut slope section should generally not vary in width throughout the section and should be based on the critical section of the slope design. The critical section should be defined by the Designer based on the rock cut slope height, geometry, stratigraphic characteristics, and anticipated rock block sizes/strengths. The height of cut slope (H) should be defined as the vertical distance from the overburden bench (or lowest 2H:1V or flatter slope of more than 10 feet in height) to the base of the slope. The catchment ditch geometry may need to be modified based on hydraulic concerns.

It should be noted that the mid-slope geotechnical (lithologic) bench is not to be designed as a rockfall mitigation measure. However, its contribution to propagate or attenuate rockfall hazards should not be ignored when defining the slope geometry during the rockfall simulation modeling. This is accomplished by evaluating both end-of-construction as well as long term conditions. This will account for surface softening and anticipated future weathering of the rock slope.

8.8 ROCKFALL SIMULATION MODELING

Rockfall Simulation Programs have been widely used in the field of geotechnical engineering since the early 1990s. All Detailed Rock Cut Slope Designs for new slopes should include rockfall simulation modeling to assess the performance of a proposed slope design. All mitigation designs for existing slopes should include at least one rockfall simulation model to assist in the selection of the appropriate rock slope mitigation methods.

A variety of computer programs are available to Designers for simulating the outcomes of a rockfall for a given slope. The two most common programs being used today are the Colorado Rockfall Simulation Program version 4.0 (CRSP) and RocFall by Rocscience. The Department has traditionally used CRSP, which provides a two-dimensional analysis of the trajectories and energies of a potential rockfall in its model. The following section provides an overview of the CRSP software, the field data collection requirements, the software input requirements, and its limitations. A detailed explanation of CRSP is beyond the scope of this publication. For additional guidance, reference the CRSP User's Manual.

Rockfall software programs, such as CRSP, divide a rock cut slope into cells or segments. Each segment is assigned a value for surface roughness, normal, and tangential coefficients based on the observed field conditions and are explained in more detail below. The following input parameters and field observations are required to complete a rockfall simulation model using CRSP:

- Slope Profile - An accurate measurement of the slope profile is required. This information can be acquired from survey data obtained by conventional survey methods, LiDAR, or digital photogrammetry methods. It is important to accurately capture any changes in slope geometry, slope geology, and locations of soil and vegetation that mantle the rock cut slope
- Rockfall Source Area – Determine the source area in which rockfall is generated. This area may represent the entire rock slope, or may include natural rock outcrops above the cut slope.
- Rock Shape – Determine the rock shape of fallen rock blocks at the base of the slope and within the source area. CRSP allows three shapes to be modeled, spherical, cylindrical, and discoidal. Spherical shaped rocks are generally chosen to represent the worst-case scenario.
- Rock Size – Determine the rock size of fallen rock blocks at the base of the slope and within the source area. If the rock cut is proposed, investigate any available natural outcrops within the study area and determine discontinuity spacing to estimate potential rock block sizes. CRSP does not allow multiple rock block

sizes to be modeled within one model run. Model the largest potential block for a conservative analysis.

- Tangential Coefficient – The tangential coefficient of frictional resistance determines how much the component of the rock's velocity parallel to the slope is slowed during impact. This coefficient is affected by slope material, slope irregularities and vegetation on the slope.
- Normal Coefficient – The normal coefficient of restitution is a measure of the change in velocity normal to the slope after impact compared to the normal velocity before impact. This coefficient is estimated by the rigidity of the slope surface.
- Surface Roughness – Surface roughness is the perpendicular variation of the slope within a slope distance equal to the radius of the rock. It describes the slope angle experienced by the rock on impact. The maximum allowable variation is calculated by determining the surface roughness (S) and the rock radius (R) using the following equation:

$$\theta_{max} = \tan^{-1}\left(\frac{S}{R}\right)$$

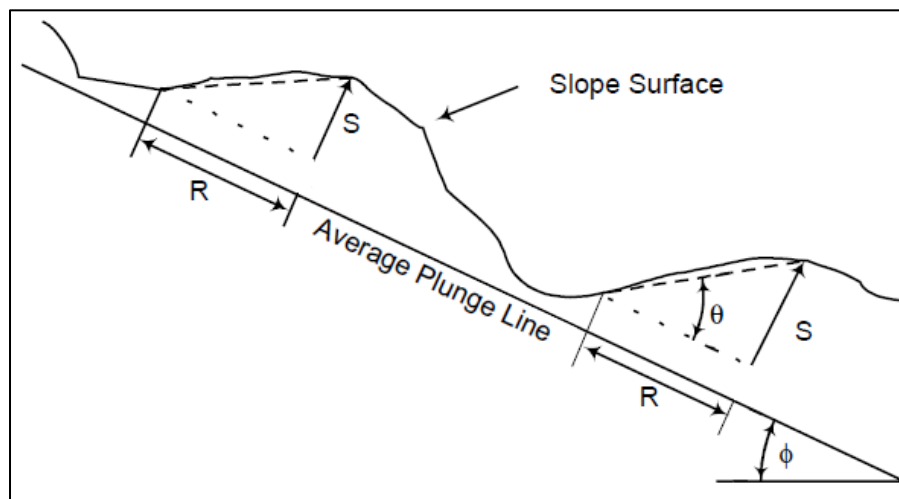


Figure 8.8-1 – Surface Roughness Diagram

[Figure 8.8-1](#) shows the range in variation as it relates to the surface roughness. The higher the surface roughness value the greater the variation in slope angle experienced by the rock on impact. The surface roughness can be estimated in the field by laying a measuring tape down the face of the slope. When the tape is pulled tight, the average distance between the tape and the slope within the distance of the rock radius is the surface roughness. The surface roughness will vary with rock size. For slopes that are not accessible, past experience may lead to the best judgement for the surface roughness.

- Location of Analysis Points – The location of the analysis points give the estimates of bounce heights, velocities, and kinetic energy. These estimates are used to design appropriate hazard mitigation systems such as catchment ditches,

and/or rockfall barriers. Typical analysis points include: top of the catchment ditch, edge of roadway shoulder, top of guiderail/barrier, and centerline of roadway.

The Department has established several guidelines to assist the Designer when using CRSP to model rockfall. If other rockfall modeling programs are used the designer should evaluate the rock and slope parameters accordingly to provide the most appropriate conditions. It should be noted that these guidelines have been largely adopted from the Ohio Department of Transportation (ODOT, April 2011) Rock Slope Design Guide with some modifications.

- Analyses should be run for both end of construction conditions and for long-term conditions to account for slope weathering.
- Analysis points are locations on the horizontal axis of a model where the resultant output variables such as energies, bounce heights, velocities, and percentage of rocks passing are summarized. A simulation model may have multiple analysis points. The Department recommends the following analysis points be evaluated, at a minimum:
 1. Analysis Point 1, or AP1, is defined as the top of the catchment ditch.
 2. Analysis Point 2, or AP2, is defined as the outside edge of the pavement (typically paved shoulder). The Department requires 95% catchment of rockfall at AP2.
- Surface Roughness accounts for the surface irregularities along segments of a slope. This value should also vary with the size of rock being analyzed. Based on the algorithm used in the rockfall simulation model, surface roughness is considered to be the most sensitive variable. For end-of-construction versus long-term conditions the following recommendations regarding surface roughness are provided:
 1. For the analysis of end-of-construction conditions, the surface roughness should be a low value (0.15-0.50 for freshly cut portions of the slopes).
 2. For long-term analysis, the surface roughness should be higher than end-of-construction analysis. Surface roughness should be adjusted based on engineering judgment using, for example, the geometry of existing slopes in similar geology.
- Rock dimensions for analysis should be site-specific. Consideration should be given to the size of fallen rocks in the existing slope ditch or in ditches in the vicinity of the site. Analysis should be performed for both the anticipated average and maximum size of potential rockfall.
- A unit weight of the Design Unit should be input based upon laboratory testing of the collected rock core samples, bulk samples, or published values where laboratory testing is not available.
- When modeling a claystone slope, use winter conditions (worst case), during which the ground is frozen resembling a “stronger” surface versus the softer conditions of spring.

- For the Normal and Tangential Coefficients, the CRSP User’s Manual provides broad ranges of values to be used for different slope conditions. [Table 8.8-1](#) may also be used as a guide for a more refined selection of initial coefficient values. It should be noted that these coefficients, which are energy dissipation coefficients, are less sensitive to the rockfall simulation, but are still important for an appropriate computer simulation.

Table 8.8-1 – Hardness Reference Guide with CRSP Coefficient Values¹

Hardness Input Code	Consistency/ Hardness	Field Identification	Normal Coefficient Values (R _n)	Tangential Coefficient Values (R _t)
Soil				
1	Very Soft	Easily penetrated several inches by fist	0.10	0.50
2	Soft	Easily penetrated several inches by thumb	0.10	0.55
3	Firm	Can be penetrated several inches by thumb with moderate effort	0.15	0.65
4	Stiff	Readily indented by thumbs but penetrated with great effort	0.15	0.70
5	Very Stiff	Readily indented by thumbnail	0.20	0.75
6	Hard	Indented with difficulty by thumbnail	0.20	0.80-0.85
Rock				
7	Very weak	Can be carved with a knife. Can be excavated readily with a point of a pick. Pieces 1 inch (25 mm) or more in thickness can be broken by finger pressure. Can be scratched by fingernail.	0.15	0.75
8	Weak	Can be grooved or gouged readily by a knife or pick. Can be excavated in small fragments by moderate blows of a pick point. Small, thin pieces can be broken by finger pressure.	0.15	0.75
9	Slightly Strong	Can be grooved or gouged 0.05 inch (2 mm) deep by firm pressure of a knife or pick point. Can be excavated in small chips to pieces about 1-inch (25 mm) maximum size by hard blows of the point of a geologists pick.	0.20	0.80

Hardness Input Code	Consistency/Hardness	Field Identification	Normal Coefficient Values (R_n)	Tangential Coefficient Values (R_t)
Rock (cont.)				
10	Moderately Strong	Can be scratched with a knife or pick. Grooves or gouges to ¼” (6mm) deep can be excavated by hand blows of a geologist’s pick. Requires moderate hammer blows to detach hand specimen.	0.25	0.85
11	Strong	Can be scratched with a knife or pick only with difficulty. Requires hard hammer blows to detach hand specimen. Sharp and resistant edges are present on hand specimen.	0.25-0.30	0.9
12	Very Strong	Cannot be scratched by a knife or sharp pick. Breaking of hand specimens requires hard repeated blows of the geologist hammer.	0.25-0.30	0.95-1.0
13	Extremely Strong	Cannot be scratched by a knife or sharp pick. Chipping of hand specimens requires hard repeated blows of the geologist hammer.	0.25-0.30	0.95-1.0

Notes: 1. Modified from Woodard (2004)

When completing rockfall simulation modeling of existing rock cut slopes, the Designer must meticulously calibrate the model to match the existing rockfall conditions as observed in the field. This is achieved by recording the dimensions of fallen rocks, measuring roll out distances of fallen rocks, measuring the surface roughness of the slope, and estimating the normal and tangential coefficients. The calibrated model achieves the highest degree of accuracy from CRSP, and is then used as a predictive tool for modeling different slope conditions, catchment zone, barriers, and rockfall sizes.

Rockfall simulations should be performed for critical sections of a rock slope. The number of critical sections for a rock cut is project-specific. At a minimum, a simulation must be done at the location of the highest vertical relief, known locations of rockfall initiation, and areas considered most susceptible to rockfall. The height of the cut slope (H) should be defined as the vertical distance from overburden bench (or lowest 2H:1V or flatter slope of more than 10 feet in height) to the base of the slope. Additional sections may be required due to changing slope conditions (e.g., change in elevation of roadway, etc.) or where geometries change that warrant additional simulations. Engineering judgment should be used to determine the number of simulations.

The Designer must be cognizant of two limitations within CRSP when modeling rockfall. These include:

- Failure to allow for vertical or overhanging sections of slope in the slope profile. If these exist in a slope, the section needs to be modeled slightly less than vertical.
- Failure to account for rock block breakage during the rockfall event by assuming the rock block remains intact.

8.9 HAZARD MITIGATION SYSTEMS

When it is determined that a rockfall catchment area will not adequately meet the project requirements, a rockfall hazard mitigation system is required. There are many rockfall hazard mitigation methods available to the Designer when evaluating a particular slope. The selection and design of rockfall hazard mitigation systems should be based on the following factors:

- The constructability of the system, which includes access (e.g., the selection of equipment for the on-slope construction activities, staging and maintaining work areas, etc.), sequencing (i.e., stabilization measures defined for the project performed in a specific sequence as defined in the contract plans and specifications), and site safety (i.e., required temporary protection measures under live traffic conditions).
- Available right of way
- Environmental permitting
- Public impacts – road closures and business impacts
- The level of protection required from the system
- The desired design life of the system
- Future maintenance access/repair.

Multiple types of mitigation may be required to meet mitigation goals. The rock slope conditions and results of the kinematic and rockfall simulation analyses will dictate the appropriate technique or combination of techniques for a particular rock cut slope. Generally, mitigation measures fall into the following four categories: rock removal; rock reinforcement; drainage; and protective measures and are discussed in the following subsections.

8.9.1 Rock Removal

Rock removal includes slope restoration, trimming, and scaling. With time, the face of the rock cut can weather and deteriorate to the point where rock fall reaches an unacceptable level. The volume of rock removed must be sufficient to accommodate drilling and excavation equipment.

Rock slope trimming or blast scaling is the process of removing overhanging, potentially unstable rock or a potential launch feature that can affect the trajectory of rockfall. Typically, trimming is achieved with the use of explosives where closely spaced holes may be drilled and loaded along the plane of the desired slope. Trimming is conducted on sections of a rock slope that are too large to scale using hand tools. Not all observed overhangs require removal.

Rock slope scaling is the process of removing loose rock, soil, and vegetation on the face of a slope using hand tools and is one of the most common rock removal procedures. Scaling is performed in a sequence from the top of the slope to the bottom, typically by means of rappelling. Tree removal is also included in the scaling process. In some cases, air bags are placed in open joints and inflated to remove hazardous rock blocks from the slope. Scaling is an effective short term solution and less favorable for a long-term solution due to the ongoing weathering and relaxation of the slope face. Often scaling is used in conjunction with another suitable protective measure such as a rock fall fence, catchment ditch, or anchored wire mesh.

8.9.2 Rock Reinforcement

Rock reinforcement includes rock bolts, rock dowels, shotcrete, buttresses, shear pins, and cable lashing and meshes. Rock bolts and shotcrete are the most common types of rock reinforcement. Typically, rock bolts are used where the rock blocks being stabilized are more than 3 to 6 feet thick or where there has been previous movement. To increase resisting forces, rock bolts are anchored in stable rock behind the failure plane creating tension and increasing the normal force on the plane. Determination of the bond length (the length of the bolt that is grouted) behind the block to be stabilized depends on the rock type and whether the anchorage is mechanical, or set with a resin or cement grout. Rock bolts can be used to stabilize individual rock blocks and entire rock cut slopes ranging from ten to several hundred feet. Rock bolts are comprised of rigid steel bars ranging in size from about 5/8 inch to 2 inches in diameter and up to approximately 100 feet in length. The surface of the bar is typically corrugated, to improve the steel grout shear strength, and epoxy coated, to provide corrosion protection. The exposed end of the rock bolt is often threaded so that a bearing plate and hexagonal nut can be installed. This will hold the applied tension and transfer the load from the bearing plate to the rock. Rock bolts are encased in resin or cement grout which provide corrosion protection and transfer the bolt tension load to the surrounding rock mass.

Unlike rock bolts, rock dowels provide no tensional resistance until the rock mass movement occurs. Dowels are used for passive reinforcement and are commonly installed before rock mass movement. Once rock block movement occurs, the tensile strength of the dowel is mobilized. The dowel resists the sliding, and the normal force between opposing discontinuity surfaces increases shear resistance along the discontinuity. Furthermore, dowels are typically constructed on new rock cut slopes to prevent relaxation and block movement related to excavation. Rock dowels are less expensive than rock bolts because they are faster to install and require one mobilization to each dowel location.

Shotcrete is a Portland cement concrete product containing fine grained aggregate that is sprayed onto a rock slope with compressed air and a nozzle applicator to form 3 to 4 inch thick layers. It is best applied to slopes containing closely fractured or highly degradable rock. The shotcrete can control both the fall of small rock blocks and prohibit undercutting of more competent lithologic units by providing protection to underlying more erodible lithologic units. The primary function of shotcrete is surface protection, by itself shotcrete provides little tensile strength for reinforcement. The effectiveness of shotcrete generally depends upon the condition of the rock surface during the application. The exposed rock surface should be free of loose or

broken rock, soil, vegetation, and ice. It is important that drainage panels and weep holes are installed behind and through the shotcrete to prevent buildup of hydrostatic pressure behind the face. Weep holes are typically 1.5 feet deep and 3 to 6 feet apart. A wire mesh is utilized to reinforce shotcrete for permanent applications. It is important the wire mesh be completely encapsulated in shotcrete to prevent corrosion of the wire mesh. Shotcrete may be done in conjunction with rock bolting to increase both global stability and surface durability of the rock slope.

Appropriate rock bolting and shotcrete design specifications for rock slope stability are developed based on site specific geologic conditions. Specific design methods are beyond the scope of this chapter. The Designer may reference Section 1043 of Publication 408 for shotcrete specifications and rock anchor bolt specifications associated with rock slope stabilization are typically defined under special provisions.

Other less common types of rock reinforcement include buttresses, shear pins, cable lashing systems, and anchored mesh systems. Buttresses are typically comprised of cast-in-place concrete or reinforced steel that provide support to overhanging rock, or provide lateral support to a rock face. Whenever possible, buttresses should be placed in compression where bending moments and overturning forces are minimized. Shear pins are composed of grouted steel bars and provide shear support at the leading edge of a dipping rock block or slab. Shear pins could be appropriate when stabilizing a large individual rock block located on a basal sliding surface such as a bedding plane or foliation plane that cannot be removed from the slope safely.

The most basic cable lashing system is comprised of two anchors and a tensioned cable passed in front of a rock block. A key design consideration is the shape of the rock block and the ability to wrap the cable tightly around the front of the rock between the anchor points. Cable lashing is useful in stabilizing rock on a fairly stable slope where removal of the block by scaling is not feasible.

Draped and anchored mesh systems use either woven wire fabric, such as chain link, gabion, or high-tensile strength, steel mesh or cable nets. These systems are utilized on slopes where there is a limited rockfall catchment area at the base of the slope. While draped mesh allows the rocks to fall to the bottom of the slope where they can be removed periodically, anchored systems effectively trap the dislodged rock against the slope. One potential disadvantage of an anchored mesh system is that if rocks fall or become dislodged, they will stay behind the mesh and without proper maintenance, the amount of rock can exceed the capacity of the mesh, creating a failure within the system. In addition, maintenance of an anchored mesh system can be difficult and dangerous as periodic disconnection of the mesh from the anchors is required to release loose rock from behind the mesh. Also, the Designer must consider snow and debris loads on the anchors and mesh. Draped mesh systems are best suited for rock blocks less than 3 feet in diameter and anchored systems, when designed appropriately can be used to restrain larger blocks and rock masses.

8.9.3 Drainage

Water within the rock mass contributes to slope instability and must be considered when designing rock mitigation systems. The lateral force water exerts in a tension crack increases by the square of the height of the water in the crack. Furthermore, water trapped in the slope can freeze and throughout the freeze thaw cycle, the ice can jack a block from the slope. Keeping the slope drained reduces or eliminates ice jacking. Also, drainage of surface and groundwater increases the stability of a rock cut slope and reduces the potential for rockfall. Types of drainage control include the construction of a diversion ditch at the top of the slope (refer to [Section 8.10.1](#)) and subsurface drainage control which can be achieved by drilling drain holes. Drain holes are typically installed along the toe of the slope or in rows to achieve the desired reduction in hydrostatic pressure. The drain holes must intersect fractures that transmit water to be effective. Typically, drain holes are drilled above the horizontal by 3 to 5 degrees to maintain positive draining and are self-cleaning. Spacing of the drain holes are based on the spacing and orientation of the predominant joint sets identified. Generally, the depth of the drain holes is at least 1/3 of the slope height. Drain holes can be left uncased in competent rock but should be lined with perforated PVC casing through erodible rock to minimize migration of fines.

8.9.4 Protective Measures

Protective measures, other than the rockfall catchment ditches discussed in [Section 8.7](#), include barrier systems (i.e., rigid barriers, flexible catchment fences, and drapery systems).

Rigid barriers encompass structural walls, gabion barriers, and concrete “Jersey” barriers. Structural walls such as masonry walls and soldier pile walls are very rigid and comprised of thin wall sections and due to their inability to deform to dissipate impact energy are best suited in relatively low impact capacities. Concrete “Jersey” barriers are best employed to control low impact energies (less than 30-foot-tons) and bounce heights (less than two feet) and are most commonly used to enhance the effectiveness of a catchment ditch.

Flexible catchment fences and attenuators are comprised of cable or net panels suspended on posts anchored to foundations, and tieback and lateral ropes often fitted with dissipation devices or brake rings. Stopping a falling rock requires the fence to deform and dissipate the kinetic energy of the block. The flexibility of the fence provides an advantage over rigid barriers due to reduced cost and simpler design effort due to well tested proprietary systems. Flexible fence systems are available in energy ratings from 5 to 3,000 foot-tons. The site conditions must have the ability to accommodate flexure of the fence barrier system during impact.

Drapery systems typically include a row of ground anchors installed at the top of the slope that are connected to a horizontal cable from which a woven steel mesh is suspended down the slope. The drapery system allows rockfall to occur between the slope and the mesh but directs rockfall into a catchment area by controlling its trajectory.

8.9.5 Hazard Mitigation System Limitations

It is suggested that the Designer review the considerations included in the Transportation Research Board Publication “Rockfall Characterization and Control” (Turner and Schuster, 2012) when considering rockfall hazard mitigation systems. The use of hazard mitigation systems and selection of system components should be reviewed with the DGE early in the design process so that cost and aesthetic considerations are adequately discussed.

[Table 8.9.5-1](#) provides a summary of the most common types of rockfall mitigation measures along with their limitations.

Table 8.9.5-1 – Summary of Hazard Mitigation Systems

Mitigation Category	Mitigation Measure	Limitations
Rock Removal	Slope Restoration	May have right-of-way of limitations
	Trimming	Difficulty drilling, debris containment
	Scaling	Temporary measure requiring repeated application
Rock Reinforcement	Rock Bolts	Less suitable on slopes comprised of small blocks, difficult to access slope
	Rock Dowels	Passive support, requires block movement to develop tension
	Shear Pins	Need to provide contact (cast-in-place concrete) between bars and rock block
	Shotcrete	Wire mesh or fiber reinforcement required, poor aesthetics unless sculpted, reduction in slope drainage, does not provide global support or resist large block movement
	Buttress	Slope height limitations, tend to be costly to construct
	Cable lashing	Typically, movement must occur for full cable resistance to develop
	Anchored high tensile-strength steel mesh systems	Rockfall debris can accumulate in pockets between the slope face and mesh, difficult to clean out
Drainage	Weep drains	Difficult to quantify need and verify results
Protection Measures	Draped mesh or cable nets	Requires a debris collection ditch area, must consider snow and debris loads on anchors
	Rigid barriers	Barriers are more prone to damage by higher energy events, can complicate cleanout
	Flexible barriers	Require room for barrier to deflect during impacts, must be cleaned out periodically.

8.10 SPECIAL GEOTECHNICAL CONSIDERATIONS

In the design of rock cuts, situations may arise where modification to a design template is necessary. This section discusses several of the more common geotechnical situations that are encountered in the Commonwealth. Design of these or other unique cases should be done using sound engineering judgment and close coordination with the DGE.

8.10.1 Surface Water

Drainage along the top of a cut slope must be addressed to minimize the amount of water flow across the cut slope face. Drainage control measures should be designed to address site specific flows and velocity. Surface water may enter fractures in the rock increasing the water pressure within the rock slope, or travel down the slope causing more rapid degradation of less durable materials. Therefore, where surface water is expected, it is often beneficial to install a diversion ditch behind the crest of the slope and on individual benches. Ditches should be hard-armored with concrete or slush-grouted aggregate to protect against erosion and degradation based on the flow velocities anticipated.

8.10.2 Underground Openings and Seals

Underground openings such as caves and mines encountered in rock slopes have the potential to destabilize portions of the slope and cause rockfall or deep-seated failure. Therefore, efforts should be made to confirm the presence and conditions of underground openings during the rock slope office and field reconnaissance. The impacts of underground openings should be considered in the location of benches.

Openings that could destabilize the slope should be cleared of debris by excavation equipment and then backfilled a limited horizontal distance into the opening with large aggregate or suitable structural support. Where the opening drains towards the slope face, a drain should be installed to prevent impounding water behind the backfilled material. If there is a potential for buildup of gas due to the placement of backfill or structure barrier, consider a ventilation system. Where voids are encountered, the stabilization methods should be evaluated for the long-term site conditions.

8.11 ROCKFALL REMOVAL CONSIDERATIONS

Quick field decisions regarding rockfall removal, temporary stabilization methods, and barrier placement prior to implementing a permanent rockfall mitigation design may be necessary when responding to a rockfall event. The extent of rockfall cleanup and temporary stabilization response will need to be determined on a case-by-case basis and, in general, there are no rules of thumb. The Department recommends retaining a professional geologist with structural geology and rock slope engineering experience to evaluate the rockfall event and provide onsite recommendations for rockfall removal and temporary stabilization methods.

8.12 CONTRACTOR SPECIFICATION CONSIDERATIONS

Rock cut slope mitigation typically requires the services of a specialty subcontractor with proper qualifications that meets extensive experience requirements based on the work tasks and complexity of the rock slope mitigation.

Most rock cut slope mitigation requires the development of a special provision specific to meet the design specifications. Rock cut slope stabilization projects are dynamic. During the construction mitigation phase of the project, the limits of treatment and the quantity of pay items will change. Therefore, special provisions should include a contingency clause for pay items, such as rock anchors and anchored mesh, as these quantities may increase based on varying slope conditions encountered during construction. In addition to a contingency clause, specifications should clearly identify any temporary rockfall protection systems that are required to be implemented during construction. Typically, temporary protection systems are contractor designed based on the means and methods of the contractor.

It is very important to provide special provisions which provide both well-defined intent (specifications) and illustrations (plans and details).

Some special provisions commonly required in rock cut slope mitigation are:

- Rock Slope Scaling
- Clearing and Grubbing
- Slope Preparation for wire mesh
- Rock Anchor Bolts
- Shotcrete Facing
- Temporary Rockfall Protection
- Horizontal Slope Drains
- Rockfall Fence
- Anchored Wire Mesh Slope Treatment
- Slope Drapery

In addition to the special provisions listed above, commonly used geotechnical details are listed below:

- Shotcrete Facing Details
- Anchored Mesh Bolt Details
- Anchored Wire Mesh Facing Details
- Horizontal Drain Details
- Rockfall Fence Details
- Slope Drapery Details

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CHAPTER 9 – COMPACTION OF EMBANKMENTS AND FILLS

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2018
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 10 – ACID BEARING ROCK

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10.1 INTRODUCTION

This chapter of this publication provides guidelines, recommendations, and considerations for the investigation, testing, identification and treatment of potential Acid Bearing Rock (ABR) in highway construction projects. Some unconsolidated overburden materials (soils) and fills may also be a potential source of acidity. When there is indication of the potential for soils to produce acid, the same processes (geochemical testing, analysis and mitigation) and criteria apply, however sample collection techniques may differ.

10.2 UNDERSTANDING ABR

10.2.1 Hazards of Acid-Bearing Rock (ABR)

In addition to presenting abrupt and adverse environmental concerns, exposed acid-generating earth material can also have damaging long-term effects on highways and highway structures. Potential geotechnical and environmental problems include:

- Corrosion of concrete and steel structures
- Destabilization of cut-slopes and fill-slopes
- Ground heaving of structures and pavements
- Toxicity to roadside vegetation
- Toxicity to aquatic life
- Degradation of drinking water supplies

The environmental concerns presented by excessively acidic leachate are only worsened by the ability of the acid to dissolve excess metals such as iron, aluminum, and manganese from the host rock and soils. Excess concentrations of these dissolved metals can readily develop in the acidic drainage, which then becomes potentially even more harmful to vegetation and aquatic life.

10.2.2 Sulfide Minerals

The primary source of acidity in Pennsylvania sedimentary rocks is sulfide minerals, with pyrite (ferrous disulfide - FeS_2) being the major contributor. Although pyrite minerals are not always large enough to be visible to the un-aided eye, larger crystals have a yellowish, metallic appearance. Deposits containing pyrite concentrations greater than 0.5% have the potential to be significant sources of acid. Various other forms of sulfide minerals are of lesser concern due to their chemical stability, and include chalcopyrite (CuFeS_2), galena (PbS) and sphalerite (ZnS), but can be problematic when present with pyrite.

Although there are more than 200 common minerals that contain sulfur, only those classified as iron sulfide are of potential concern due to the ability of these elements to promote oxidation, hydration and the release of acid. In Pennsylvania, there are four potential sulfide deposits types, listed as follows in descending order of pyrite oxidation reactivity:

1. Veined Rock Deposits (High oxidation potential)

These “epigenetic” sulfide compounds were formed in the existing discontinuities to form veins within the rock deposits, and are a result of hydrothermal intrusions of deep, hot, mineral-rich water. These hydrothermal sulfur minerals tend to be highly concentrated in random rock joints and fractures, are much less common than sedimentary deposits, and are usually much more difficult to detect. Any project site suspected of involving sulfide deposits of hydrothermal origin requires more thorough consideration and potentially higher level of subsurface investigation. For such sites, a reasonable effort is to be taken to search for and review available on-file geologic reports, as further described in [Acid-rock Drainage at Skytop, Centre Co., PA, 2004 \(Open-File Report 2005-1148\)](#)

Although these Department ABR management guidelines focus primarily on the identification and treatment of the more common sedimentary sulfides, due consideration must be given to other potential sulfide sources, including, when identified, necessary treatment and mitigation measures. If there is existing documentation indicating veined deposits may be present on a project site, or if field reconnaissance and/or subsurface investigation activities provide strong evidence of their potential presence (even though not specifically identified), project-specific contingency actions should be developed for ready for use during construction, if such materials or conditions are encountered. In such instances, qualified personnel capable of identifying such deposits should be on site during any excavations of the suspect locations. If an epigenetic condition (veined deposit) is positively detected, specifically mapped, or previously identified, reasonable and appropriate measures must also be taken to address this during design, and included as part of the contract items, plans and provisions.

2. Sedimentary Rock Deposits (Moderate oxidation potential)

These “syngenetic” sulfide compounds were formed entirely from the original sediments, and are now disseminated throughout rock deposits. Since sedimentary sulfur is deposited relatively uniform across the original sediments, it is relatively easy to detect by investigating across (perpendicular to) the planes of the bedding. Common syngenetic ABR in Pennsylvania includes coal and black shale deposits.

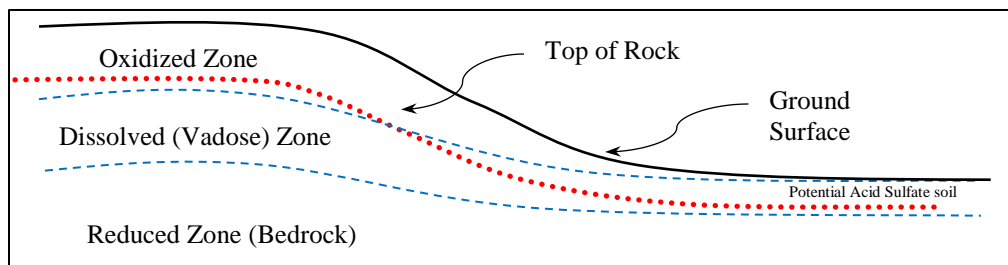


Figure 10.2.2-1 – Generalized Oxidation Profile

3. Mine Spoils (Moderate Oxidation Potential)

Mine spoils are previously disturbed materials which typically have erratic zones and pockets of fill with varying sulfur contents. The extent of these zones is more difficult to accurately define than that of naturally-occurring, stratified rock deposits. When old mine spoils

need to be excavated for highway construction, it is best to apply alkaline material at a higher rate than normal. This is achieved by employing the following:

- Consider all sulfur contents measured in the lab testing as potentially acid-producing (ignoring the $\geq 0.5\%$ sulfur indicator) and add supplemental alkaline material accordingly.
- Set the target NNP value of the mine spoil equal to 20 parts per thousand (ppt) CaCO_3 . This is the upper limit of the normal target NNP range of 12 - 20 ppt.
- Maintain a factor-of-safety of two (F.S. = 2) when calculating supplemental alkaline addition rates (refer to [Section 10.7.4](#) for further details).
- Anticipate that existing spoil material will be excavated and managed in lifts no greater than 20 feet. This will enable proper handling and alkaline treatment of the spoil material in most situations.

4. Acid Sulfate Soil Deposits (Low oxidation potential)

These “supergene” sulfate compounds can be formed within low-lying residual soils, mine spoils, or fills in which dynamic fluctuations in groundwater levels (vadose zone) create conditions favorable to the deposition of acids and dissolved salts within the soils and weathered rock.

10.2.3 Sulfuric Acid Formation

Oxidation of pyrite and other sulfide minerals result in the formation of sulfuric acid. The oxidation process occurs in the presence of oxygen and water. In the case of pyrite for example, the summary reaction is:



The complete process is a multi-step reaction, with each step resulting in more acidity. As the acidity increases, this feeds the reaction, and the process becomes a run-away reaction. As the acidity increases, and the pH drops below 6, acid-loving bacteria, *Thiobacillus Ferroxidans*, begin to thrive. These bacteria are widespread in the environment, and readily oxidize sulfide minerals. The presence of these microbes can increase the rate of sulfide oxidation from 50 to one-million times.

In addition to the concentration of sulfide minerals, other factors that impact the rate of acid production include temperature and particle surface area. Heat is a catalyst for the reaction, and the bacteria thrive at a temperature range of 68F to 95F (20C to 35C). Size and mineral crystal structure control the surface area available for oxidation. As surface area increases, the rate of acid production increases. Therefore, fine-grained framboidal pyrite particles with diameters of 1μ or less, and also long needle-like crystal-shaped pyrite, will oxidize more rapidly than coarse-grained particles with diameters exceeding 50μ . In undisturbed (non-excavated) material, the much lower availability of oxygen, keeps the reaction in check. The oxidation

process occurs over many years, keeping acidity in groundwater at very low levels, and preventing toxicity.

10.2.4 Oxidized Cap Rock

It should be understood that the formation of acid from ABR is a common process that occurs slowly in the natural environment. There is usually a “cap rock” of oxidized material above any ABR. This cap rock is commonly termed “Oxidized Cap Rock”, or OCR. The oxidized material produced acid under natural conditions, but at a greatly reduced rate that did not result in concentrations causing environmental toxicity. The process took many years in an undisturbed state that limited available oxygen. The limited oxygen availability controlled the production of acid. Once disturbed, the ABR material is in a much different atmosphere, with greater surface area and much more abundant and available oxygen.

The presence of this OCR zone can provide both an indication of the potential existence of ABR below, and a potential means of addressing the problem. The thickness of the oxidized zone at any given site can be variable and irregular. In Pennsylvania, OCR thickness ranges from zero to over 80 feet, with a typical thickness of around 20 feet. The oxidized zone can usually be visually determined by strong red, orange, or white colored metal oxide staining on the jointed surfaces of rock core samples. In addition, a friable, weathered rock indicates oxidation. Knowing the limits of the oxidized zone presents an opportunity to simply avoid disturbance of potential ABR, by limiting excavations into the material where practical, and still meet project objectives.

10.3 SITE ASSESSMENT

10.3.1 Site Assessment Protocol

Many highway construction projects require a considerable amount of earth excavation. A sufficient and reasonable level of effort needs to be given during project design to determine if acid-bearing rock and/or soil exist at the site. If ABR is present, it is then important to adequately define the type and extent of the sulfide/sulfate deposits. With this knowledge, earth excavations can be planned properly. The general protocol for planning and management of ABR is outlined as follows:

1. Review site-specific geologic literature and mapping
2. Perform reconnaissance field survey
3. Contact the appropriate PA-DEP Regional or District Mining Office for possible guidance on site geology and sampling strategy suggestions
4. Develop sampling strategy
5. Perform exploratory drilling
6. Complete background groundwater and surface water sampling and laboratory analysis
7. Determine composition and species of minerals
8. Perform supplemental exploratory drilling and sampling if needed
9. Perform laboratory testing of rock and soil samples
10. Map subsurface geochemical trends relative to planned excavation

11. Complete the Geotechnical Engineering Report that incorporates all information from the previous ten steps
12. Develop final project construction earthwork plans and specifications to address the acid-bearing rock and soil deposits to be encountered. This may require Special Provisions and/or design drawings to detail requirements for ABR material handling, disposal, and/or treatment.
13. If Acid Rock Drainage (ARD) is anticipated, and the treated discharge is discharged into waters of the Commonwealth a National Pollutant Discharge Elimination System (NPDES) permit is likely required. Contact the appropriate PA-DEP Regional or District Mining Office for assistance with the permitting process.
14. Contact PA-DEP Regional Office for review of ABR management plan
15. Monitor and document site conditions during site excavation and ABR treatment construction
16. Periodically monitor and document post-construction site conditions for a period of three years, or more if indicated by the results of water quality monitoring or if dictated by PA-DEP permit requirements.

10.3.2 Literature Search

The initial step in assessing a site for ABR is to research the applicable geologic literature and mapping that pertains to the project site. Consult the Pennsylvania Department of Conservation and Natural Resources – Geologic Survey (PA DCNR-GS) Open-File Report OFMI-05-01.1, “[Geologic Units Containing Potentially Significant Acid-Producing Sulfide Minerals](#)”.

This report contains both a map and text providing technical and geologic guidance in identifying potential ABR formations, and is a valuable first step in assessing the potential for ABR. Other good sources of information include Pennsylvania Department of Environmental Resources (PA-DEP), especially the District mining offices, and the United States Geologic Survey (USGS).

10.3.3 Field Reconnaissance

The site field reconnaissance, also referred to as field screening, involves walking the entire site, examining surface features for signs of ABR. These signs may include:

- Iron oxide staining or effervescence on exposed rock surfaces
- Oxide staining in streams (orange-red streambed sediments)
- Sterile streams (very clear water but no aquatic life)
- Sparse or unhealthy vegetation (low pH impacts nutrient uptake and disease resistance of many plant species)
- Outcrops of coal or and black shale
- Evidence of past surface mining or deep mining
- Efflorescence (soft powdery crystals of sulfate salts) on exposed rock surfaces

Talking to local residents or landowners about the site history may yield valuable insight into the characteristics of the site.

10.3.4 Planning the Subsurface Exploration Program

If the site assessment, or any information obtained during project design, indicates a potential for the presence of ABR, then a comprehensive ABR site investigation shall be performed as part of the geotechnical engineering report preparation. Once the anticipated site geology is defined, the locations, depths, and type of sampling will need to be determined based upon the extent of excavation anticipated for the project.

Note that [Table 10.3.4-1](#) provides the minimum requirements for a preliminary ABR site investigation (borings and testing). These borings are in addition to the geotechnical engineering borings conducted for site characterization and design. If initial laboratory testing and Acid-Base Accounting calculations indicate the presence of substantial ABR, then additional borings and testing must be considered to further delineate and quantify rock strata containing acidic bearing minerals and rock strata containing sources of alkalinity. Rock containing less than 0.5% sulfur may generate significant acidity. Rock which is lacking neutralization potential (less than 30 ppt CaCO₃) increases the potential for acidity due to poor neutralization potential. Refer to [Section 10.7](#) for more detailed information regarding the interpretation of testing results.

Thoroughly characterizing the site ABR conditions will optimize the potential costs of mitigation and treatment programs. More importantly, accurate site characterization will lower the risk that the planned mitigation and treatment measures will be inadequate. The cost of additional borings and testing necessary to thoroughly characterize the site is only a fraction of the cost of treatment and mitigation requirements. An inadequately characterized site will ultimately result in a more costly project to complete. It has been the experience of the Department that disruptions during construction due to inadequately defined subsurface conditions will impact project costs far exceeding the comparative cost of adequate investigations and well-designed mitigation/treatment plans prepared in the design phase.

The aforementioned term “substantial” ABR must be considered within the context of the anticipated methods of mitigation and treatment of ABR, and potential environment impacts of the Acid Rock Drainage (ARD).

If the planned project excavations appear to have potential for developing unavoidable ARD, the receiving watercourse classification should be determined in accordance with PA Code 25, Chapter 93, Water Quality Standards. The classification and baseline chemistry of the watercourse will play an important role as treated ARD will need to meet water quality standards established during the permitting process. For example, treated ARD may not be permitted to enter a High Quality Cold Water Fisheries stream. The level of ARD treatment required will depend on the receiving stream baseline chemistry, the volume and chemistry of the ARD discharge, and treatment methods. The level of treatment required will be a condition of the NPDES permit and/or any other permits required to discharge treated ARD.

For any project involving ABR it is important to contact the PA-DEP District office and the Department District Environmental Manager in the earliest stages of the project to gain a preliminary understanding of the ARD discharge requirements. It is also essential that a qualified environmental specialist that is experienced in treatment of ARD be included as part of the design team.

Table 10.3.4-1 – Minimum Boring Requirements for ABR Assessment⁷

Type of Project Excavation	RISK ASSESSMENT ^{3,4,5,6}		
	No ABR Risk based on site reconnaissance	ABR Risk with horizontal or slightly dipping geologic deposits	High ABR Risk or ABR Risk with steeply dipping or complex geologic deposits
Roadway Cuts (typically Class-1 excavation) or Retaining Walls (typically Class-3 or Class-1 excavation)	No additional borings required.	For $L \geq 300$ ft.: $N_b = \frac{L}{500} + 2$ For $L < 300$ ft.: $N_b = 2$	For $L \geq 200$ ft.: $N_b = \frac{L}{300} + 2$ For $L < 200$ ft.: $N_b = 2$
Structure Foundations (typically Class-3 excavation <200 ft. in length or width)	No additional borings required.	$N_b = 1$ (per structure)	$N_b = 1$ (per substructure)
Drainage Facilities (typically, Class-3 or Class-1 excavation >200 ft. in length or width)	No additional borings required.	For $L \geq 500$ ft.: $N_b = \frac{L}{500} + 1$ For $L < 500$ ft.: $N_b = 2$	For $L \geq 300$ ft.: $N_b = \frac{L}{300} + 1$ For $L < 300$ ft.: $N_b = 2$

- Notes: 1. (N_b) is the minimum required number of borings rounded to the nearest whole number.
2. (L) is the length of cut-slope, wall, or drainage facility excavation.
3. As discussed below, local geologic conditions must be considered in the selection of individual boring locations, spacing, angles, and depths.
4. The completion of the minimum required exploratory borings as outlined above may reveal that the stratigraphy and structure of the rock formation require additional borings to investigate all strata and materials present. If geologic contacts or discontinuities are encountered that may yield varying and adverse conditions across the site, then additional borings may be necessary to investigate all potential sources of ABR.
5. An adequate number of borings should be obtained to enable the strata to be correlated on geologic cross-sectional plots. The drilling program must attempt to

adequately sample all suspect strata and materials to be excavated during construction, as well as cut-surfaces (such as subgrades, foundations, and slopes) to be left temporarily or permanently exposed. Boring depths shall advance to the deepest anticipated limit of excavation, plus five feet.

6. Vertical borings are typically sufficient for exploration of flat-bedded geology. For steeply dipping rock or other complex geology, inclined borings may be necessary to sufficiently explore a sequence of deposits as shown in [Figure 10.3.4-1](#). The location and spacing of borings are dependent upon the orientation of the anticipated sulfide deposits. Near-vertical deposits will likely require a shorter boring interval than gently dipping deposits. Angled borings are usually more difficult and costly to obtain than vertically oriented borings. Boring inclinations up to 30° (from vertical) are usually obtainable. Inclinations greater than 45° are not typically attempted.

7. The site field reconnaissance is considered part of the risk assessment, which may also include a Subsurface Exploration Plan Submission (SEPS) and subsequent preliminary borings. At a minimum, the SEPS is to include a thorough well-documented assessment of the site geology, field conditions (topography, hydrology) with special emphasis on identifying any signs or indicators of potential ABR. These tasks must be completed by personnel experienced in the identification of field conditions which suggest current or potential development of ABR. The reconnaissance and risk assessment must be conducted within the quality and sensitivity of the local environmental conditions, and potential risks and costs associated with potential environmental damage.

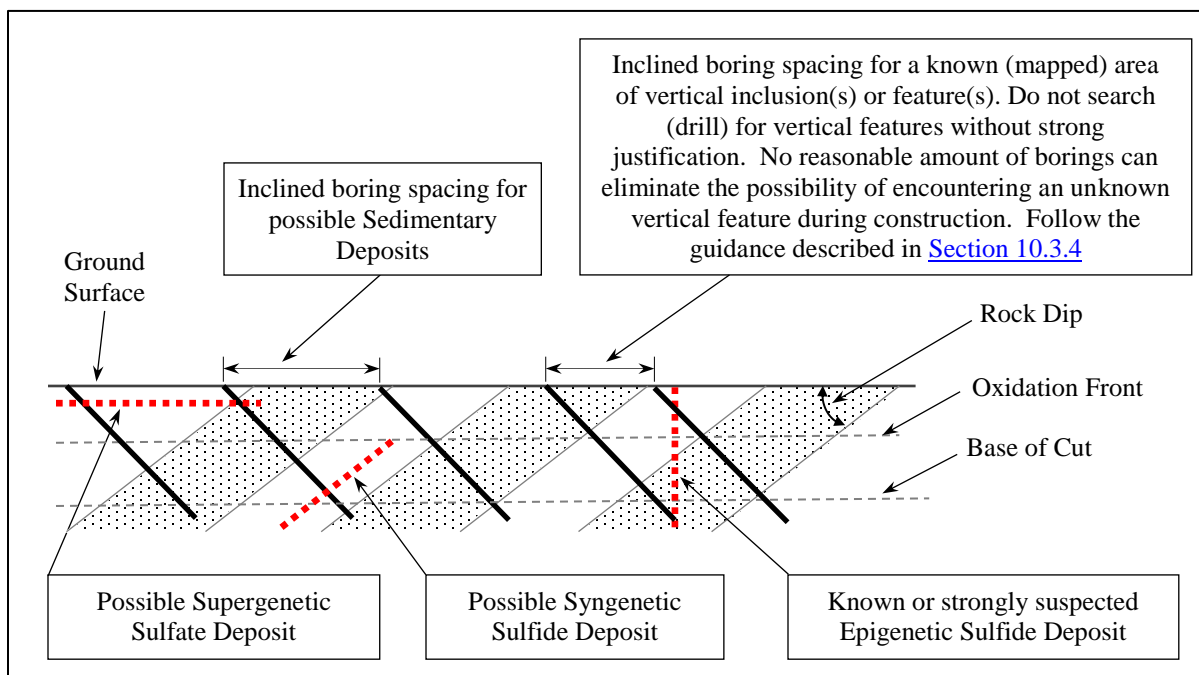


Figure 10.3.4-1 – Inclined Borings in Series to Explore Dipping Geologic Sequences

10.3.5 Prevalent Sulfide-Bearing Deposits in Pennsylvania

Various sulfide bearing rock and soil types in Pennsylvania have proven to be of greater significance related to the generation of acidic drainage. The following general lithologies should be suspected of being acid-bearing:

- Coals and coal remnants
- Black shales, carbonaceous mudstones, carbonaceous claystones
- (typically, >25% carbonaceous that create sooty black (not gray) streak)
- Sandstones with carbonaceous inclusions
- Channel sandstones and channel sands
- (regular sequence of turbidite bedding structure shown in [Figure 10.3.5-1](#))
- Mine spoils
- Isolated sulfide deposits

Note that isolated sulfide deposits are not part of the normal lithification (rock forming) process in sedimentary rock formations and therefore may be very difficult to locate precisely unless previously mapped. There is some evidence that suggests mapped geologic lineaments are host features to cross-strike epigenetic sulfide inclusions. Accordingly, mapped lineaments may give indication to a heightened possibility of isolated sulfide deposits. For this reason, projects located in areas suspected of these deposits may warrant some reasonable amount of additional exploration to attempt to identify these features during project design, with any remaining uncertainties as to the possible existence of these deposits being best addressed by provisional actions specified in construction contracts with qualified personnel present during excavation activities to promptly identify these deposits. Provisional ABR actions should be included in the construction contract only if project design activities indicated such a potential, or if reasonable concern can be supported that they may be present.

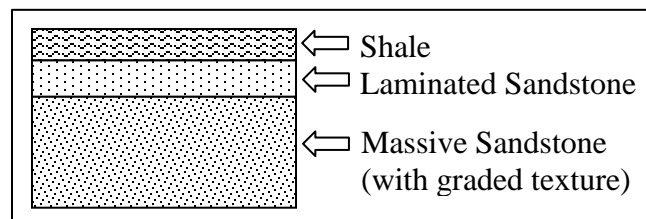


Figure 10.3.5-1 – Common Depositional Sequence of Channel Sandstones

10.3.6 Prevalent Alkaline-Bearing Deposits in Pennsylvania

Various rock types in Pennsylvania are predictable sources of alkalinity, and can serve to buffer and/or prevent the generation of acidic drainage. The following general lithologies are common alkaline sources:

- Limestones
- Dolomites
- Calcareous mudstones,

- Calcareous claystones
- Calcareous glacial tills
- Sandstones with calcium carbonate cement

10.4 SUBSURFACE INVESTIGATION

10.4.1 Sample Collection Methods

There are three common methods of sample collection used in geochemical site investigations; core drilling, air-rotary drilling, and outcrop sampling.

10.4.1.1 Core Drilling

High-recovery core borings are often a preferred method due to the ability to retrieve intact rock specimens. Important features can usually be distinguished in core samples such as fracturing, fissility, bedding plane inclinations, and fossils. In locations where hydrothermal ABR (veined deposit of sulfide minerals) is suspected, the recovery of core samples permits the visual examination and verification of these features if the deposits are encountered during drilling. For borings in complex geology that may encounter multiple joints sets having sulfide inclusions, drill core orientation measurements using a core goniometer will provide dip and strike of inclined strata and joints allowing for a more complete interpretation of geologic structure.

However, due to the isolated nature of these deposits, they are very difficult to locate with borings. For sites where such deposits are suspected based on office and field reconnaissance but not verified from the completed drilling, concerns are best addressed by provisional actions specified in the construction contract, which includes having qualified personnel present during excavation activities to identify these deposits, if present.

Core samples can also be valuable in identifying some sedimentary deposits of sulfide minerals. Obtain only continuous core boring samples using a split inner core barrel. Standard core drilling methods in weakly-consolidated deposits such as certain coals, claystones, and shales may not provide satisfactory length and quality of core sample. In these cases, higher core recovery may be achieved by using slightly larger diameter drill bits (“H” and “P” designations), adjusting the drill pressure and speed, or using a triple core barrel. Air-rotary drilling is another option. This method yields an already sufficiently ground rock sample (which must be protected from air and moisture), however vertical sample delineation is less accurate.

10.4.1.2 Air Rotary Drilling

The standard-circulation air-rotary drilling method is often advantageous due to the speed and ease of sample collection. Air drilling advances quickly through even the hardest rock to produce rock cuttings, or “chips”, which are easily collected and well-suited for laboratory testing. However, this drilling method can present some inherent limitations in providing representative samples, particularly noted as follows:

- Accurately correlating test results to specific depths and/or strata. To increase sample accuracy, drill bit advance should be halted for each sampling increment (3 feet maximum) and the hole blown clean before subsequent drilling.
- Rock chip samples shall be collected from the collection pan only. Material is not to be taken from the accumulated return material which has collected on the ground surface beneath or around the drill table.
- The entire increment depth must be represented by the sample that is sent to the lab. The return chips must be collected from the entire run (length) of each increment. A flat-bottomed pan is recommended to collect the return chips. To assure accurate representation, the entire amount of material retained on the collection pan shall be containerized. If an excessive amount of chips are collected for one increment (greater than 15 pounds (7 kg)), the pan sample can be reduced in accordance with AASTHO T-248, Method B – Quartering. The intent is to collect all material from the sampling increment, then draw the representative lab sample from the well-mixed cuttings and discard the remainder.
- In highly-fractured, voided rock deposits (such as karst), or loose, unconsolidated material (such as some fills), adequate compressed air circulation may be lost, and drill cuttings may not return sufficiently to the surface. In these conditions, reverse-circulation air drilling or core drilling methods may be more effective and should be considered.

10.4.1.3 Outcrop Sampling

On occasion, it is possible to obtain hand samples of rock from nearby existing highway or railroad cut-slopes, mining highwalls, or shallow excavations (test pits). Care must be taken not to sample material that has been exposed for a length of time, and may be weathered beyond that of material which is to be excavated during construction.

10.4.2 Sample Logging

The drilling inspector should document the pertinent physical characteristics of the rock and soil samples, and record the following minimum information for each sample: lithology, color, streak, grain size, moisture, weathering, mineral inclusions, effervescence (fizz) in dilute (10%) HCL, structural features, bedding characteristics, and fossils. High-quality digital color photographs shall be taken of all boxed cores prior to storage or shipment to the laboratory. Photo files or prints shall be included with the Geotechnical Engineering Report.

10.4.3 Sample Storage

All samples shall be properly labeled to allow for positive identification of the sample date, project, boring number, and depth increment. It is important to protect all collected samples from excess moisture. Core samples shall be transported with care so they will remain intact and dry. Chip samples from air-rotary drilling are best stored in one-gallon or one-quart size sealable plastic bags. Samples should be thoroughly dried prior storage.

10.4.4 Groundwater and Surface Water Sampling

During the subsurface investigation, groundwater samples should be collected in completed boreholes to obtain baseline data. Borehole water samples should be taken a minimum of 24 hours after the drilling is completed to prevent sampling groundwater with chemistry altered by addition of drill water. Several water chemistry parameters may suggest the presence of ABR. Generally, if groundwater water has a pH less than 5.0 and any combination of the following, ABR conditions may have developed:

- Total Fe greater than 7.0 mg/l
- Total Mn greater than 4.0 mg/l
- Elevated sulfide concentrations (detectable by rotten-egg odor)
- Elevated sulfates (>75 mg/l in groundwater, >250 mg/l in surface water)
- Elevated conductivity (>2,000 $\mu\text{s}/\text{cm}$)

In addition, a detailed understanding of the receiving waters chemistry is an important component in the design of a suitable treatment system. Background surface water samples should be collected at several locations both upstream and downstream of the anticipated point of ARD discharge. Sampling of individual springs and seeps is also recommended. Elevated sulfate level is a strong indicator of impacts due to past mining. Reference the U.S. Geological Survey Circular, "[Water Quality in the Allegheny and Monongahela River Basins, Pennsylvania, West Virginia, New York, and Maryland, 1996–98](#)."

10.5 SAMPLE PREPARATION

10.5.1 Sample Selection and Compositing

In addition to acidic materials, the alkaline and neutral materials also need to be defined in order to properly assess the overall geochemistry of a site. It is important to sample all zones that could contribute to acid and dissolved metals production as well as alkaline zones that may neutralize such products. Therefore, test all recovered rock core (the entire core run) within the designated sample limits. The maximum length of core for any one test sample is three feet. Group the tested material by sequential depth and strata. Strata thicker than three feet must be broken into two or more test samples. Test the one foot of material immediately above and below coal beds as separate units.

Compositing individual adjacent samples can reduce the number of tests required, and is permitted under certain conditions. Compositing must not, however, sacrifice the accuracy in determining the potential of strata to produce acidity or alkalinity. Multiple sequential thin strata may be grouped into lengths up to three feet, except if a specific unit is suspected of high potential acidity, then test that specific strata alone (do not combine with adjacent material). Individual samples with different color, texture or fizz rating may not be combined to create a composite. Compositing is permitted for equal amounts of up to three vertically-adjacent samples. The individual samples making up the composite must represent equivalent strata thicknesses of identical material (all same rock type).

10.5.2 Sample Quantity, Particle Size and Moisture

It is important to obtain an adequate quantity of sample in order that all tests may be performed properly. For air-rotary sampling, collect all materials returned by the drill, bagging samples in separate one ft. increments. If it is not practical to retain all the material collected for a given foot (i.e., too much material to keep) homogenize the material for that increment by thorough mixing, and retain a minimum one pound (500 gram) sample for testing. For a 6-inch diameter boring, one pound is roughly 5% of the material excavated by the drill bit in one foot of vertical travel. Each bagged sample will be crushed (if necessary) in the laboratory to allow all particles to pass the No. 60 sieve (250 μm) sieve. Samples from air-rotary drilling must be air-dried prior to containerizing, and sealed in air-tight containers.

For rock core samples, the entire length of core is required to be crushed until all particles pass the No. 60 (250 μm) sieve in accordance with AASHTO T 27. Core samples may also be split longitudinally before crushing in order to reduce the volume of rock to be processed, unless rock bedding or vertical veining is steep (relative to the direction of coring) such that a representative sample would be difficult to assure. A 12-inch length of split 2-inch diameter rock core yields approximately 1 to 1.5 pounds (500 to 750 grams) of crushed sample, which is adequate. The remaining half of the split core can be reserved for future reference or testing. Crushed and sieved particles are to be thoroughly mixed to produce homogeneous and representative samples for each sample increment.

Rock core samples will be wet upon extraction (due to drilling water). Core samples must be permitted to air dry immediately after extraction, and shall be protected from re-wetting or excessive humidity until delivery to the laboratory.

10.6 SAMPLE TESTING

Tests that must be conducted to proceed with acid-base accounting are; 1) the “Fizz Rating”, 2) Neutralization Potential (NP), and 3) Total Percent Sulfur. There are a variety of methods and modifications to these tests. The specific versions required are listed in [Table 10.6-1](#) and described below.

Table 10.6-1 – Acid-Base Accounting Test Methods

Test	Required Version/Method	Required Test Specimen passing the No. 60 sieve (AASHTO T 27)
Fizz Rating (FR)	Sobek Method	0.02 ounce (0.5 grams)
Neutralization Potential (NP)	Sobek Method (Siderite Correction)	0.1 ounce (2 grams)
Total Percent Sulfur (% S)	High-Temperature Combustion Method (ASTM D-4239)	0.02 ounce (0.5 grams)

10.6.1 Fizz Test

The purpose of the Fizz Test is to determine the quantity and strength of acid used in the NP test. The test is subjective and requires judgment on the part of the individual performing the test; however, determination of proper acid addition is important for the reliability and reproducibility of NP data.

Prepare 0.5 gram of sample consisting of material passing the No. 60 sieve. Add two drops of 25% HCl solution to the sample. Observe the reaction (if any) when the acid is added. Look for bubbling or a visual and/or audible “fizz”. If a reaction is observed, this indicates the presence of calcium carbonate (CaCO₃). Note the rate or strength of the reaction. Based upon the observed reaction, assign a “Fizz Rating” or FR for the sample as indicated in [Table 10.6.1-1](#).

Table 10.6.1-1 – Fizz Test

Fizz Intensity (Reaction)	Fizz Rating (FR)	Expected NP Range (ppt ¹ CaCO ₃ equivalent)
None	0	0 – 30 ppt (0 - 3 %)
Slight	1	30 – 90 ppt (3 - 9 %)
Moderate	2	75 – 475 ppt (7.5 - 47.5%)
Strong	3	450 – 1000 ppt (45 - 100%)

Note 1. (ppt) = parts per thousand

10.6.2 Neutralization Potential (NP) Test

The neutralization potential (NP) test provides a measure of the capacity of the material to supply alkalinity. In addition to calcite and dolomite, siderite (FeCO₃) is a common carbonate mineral found in rock associated with the Appalachian coal beds. During natural weathering, siderite initially provides alkalinity. Continued weathering of the by-products produces an acidic solution. The complete weathering process for siderite yields a zero-neutralization potential, with no net acidity or alkalinity.

The NP test however does not progress far enough to produce this net zero NP. Only the initial alkaline production is measured, overestimating the total NP of the sample. To compensate for the overestimation of alkaline, a correction is performed by adding a 30% solution of hydrogen peroxide (H₂O₂) during the test process.

10.6.2.1 NP Test Procedure:

Prepare a 2 gram of sample consisting of material passing the No. 60 sieve. Place the sample in a 250-ml flask. Add a quantity and strength of HCl solution as indicated in [Table 10.6.2.1-1](#), based upon the results of the fizz test. Add distilled water to bring the volume in the flask up to 100 ml. Prepare a “blank” flask with the same volume and strength HCl solution, topping to 100 ml with distilled water, but no crushed rock sample added.

Heat the flasks and boil gently for five minutes, then allow to cool. Gravity filter the beakers contents using a No. 40 (0.45 μm) filter. Add 5 ml of 30% H₂O₂ to the filtered solutions. Boil the solutions in flasks for another five minutes, then allow to cool. Cover tightly. Titrate the solutions using 0.1 N NaOH or 0.5 N NaOH (concentration exactly known), to pH 7.00 using an electrometric pH meter and burette. The concentration (normality) of NaOH used in the titration should correspond to the concentration of the HCl used in the previous step.

NOTE: Titrate with NaOH until a constant reading of pH 7.0 remains for at least 30 seconds.

Table 10.6.2.1-1 – NP Test

Fizz Rating	Expected NP Range (ppt ¹ CaCO ₃ equivalent)	HCL to Dispense	Range of NaOH to Dispense
0	0 - 30	20 mL of 0.1N	8 - 20 mL of 0.1N
1	30 - 90	40 mL of 0.1N	4 - 28 mL of 0.1N
2	75 - 475	40 mL of 0.5N	2 – 34mL of 0.5N
3	450 - 1000	80 mL of 0.5N	0 – 44 mL of 0.5N

Note 1. (ppt) = parts per thousand

If less than 3 ml of the NaOH is required to obtain a pH of 7.0, it is likely that the HCl added was not sufficient to neutralize the entire base present in the 2.00 gram of sample. A duplicate sample should be run using the next higher volume and/or concentration of acid as indicated in [Table 10.6.2.1-1](#).

10.6.2.2 NP Test Calculations:

1. **C** (Constant) = (ml acid in blank)/(ml base in blank)
2. **ml acid consumed** = (ml acid added) – (ml base added x C)
3. **NP** (in ppt CaCO₃ equivalent) = (ml of acid consumed) x (25.0) x (Normality of acid)

10.6.3 Total Sulfur Test

In Pennsylvania rock strata, sulfur generally occurs in three forms: sulfide sulfur, sulfate sulfur, and organic sulfur. Sulfide sulfur is the form that reacts with oxygen and water to produce acid rock drainage. Sulfide minerals include: pyrite (FeS₂), chalcopyrite (CuFeS₂), galena (PbS) and sphalerite (ZnS). Sulfate sulfur is often a by-product of the weathering of sulfide sulfur and is a potential source of acid. Organic sulfur occurs in carbon-based molecules of coal and other high carbon rocks, but occurs in compounds that are more stable and are not a contributor to acid production.

Although sulfur occurs in a variety of forms, the test methods for determining the concentrations of the individual forms are relatively unreliable. In contrast, the total sulfur

content is relatively simple to determine, and the methods used are generally very reproducible and have relatively high precision. While using total sulfur may overestimate the Maximum Potential Acidity (MPA), total sulfur methods currently provide the most reliable basis for calculating MPA. To determine total sulfur, use high temperature combustion methods (ASTM D 4239), which are the simplest and most frequently used, and provide accurate results.

10.6.4 Kinetic Testing

Kinetic test method ASTM D 5744 shall be considered during project design only if any of the standard ABA testing indicators are inconclusive (e.g., NNP = 0 – 20, or PR = 1 – 2). Kinetic testing may also be justified if the leachate chemistry of the fill or a fill mix-design needs to be quantifiably predicted.

Kinetic test methods such as ASTM D 5744 and EPA Method 1627 are laboratory weathering procedures that use alternating cycles of aqueous leaching and gaseous oxidation (saturation and draining). These laboratory weathering tests are not expected to precisely simulate site-specific field conditions; however, these tests do provide a useful measure of the leachate chemistry (pH, alkalinity, acidity, specific conductance, sulfate, etc.) of untreated ABR fill samples and alkaline-treated ABR fill.

10.7 ANALYSIS OF TEST RESULTS

10.7.1 Maximum Potential Acidity (MPA)

The Maximum Potential Acidity (in ppt CaCO₃ equivalent) is determined as follows:

$$MPA = (Total \% Sulfur)(31.25)$$

10.7.2 Potential Ratio (PR)

The Potential Ratio (PR) is the ratio between the Neutralization Potential (NP) and the Maximum Potential Acidity (MPA):

$$PR = \frac{NP}{MPA}$$

10.7.3 Net Neutralization Potential (NNP)

The Net Neutralization Potential (in ppt CaCO₃ equivalent) is the difference between the Neutralization Potential and the Maximum Potential Acidity. Positive values (+) are net alkaline, negative values (-) are net acidic:

$$NNP = NP - MPA$$

10.7.4 Interpretation of Results

[Table 10.7.4-1](#) provides guidance for interpretation of test results and analyses. Strata that have concentrations of sulfur of more than 0.5 percent may generate significant acidity. Rock materials that have a Neutralization Potential (NP) greater than 30 ppt CaCO₃, and show a reaction during the fizz test, are significant sources of alkalinity. A Potential Ratio (PR) of less than one will likely be acidic, and greater than two should be alkaline. In between one and two, the material may be either acidic or alkaline. A Net Neutralization Potential (NNP) less than zero will most certainly produce acidic conditions. NNP values between 12 and 20 ppt will likely produce alkalinity. Material with an NNP value of 30 ppt is confidently expected to generate alkalinity. On this basis, the initial target NNP level in the ABA calculations is set between 12 and 20 ppt CaCO₃, with a subsequent factor-of-safety also applied (F.S.= 2). This equates to a final target NNP value between 24 and 40 ppt. The additional quantity of SAM incorporated due to applying the factor-of-safety helps to assure that adequate lime material is available within the amended fill. The factor-of safety guards against the effects of imperfect mixing/blending, and accounts for some variability in the rock material that may occur during construction. Refer to [Section 10.8.2](#) for specific worked examples.

Table 10.7.4-1 – ABA Assessment²

Property	Threshold ¹	Interpretation
Percent Sulfur (% S)	> 0.5 % S	May generate significant acidity
Neutralization Potential (NP) and Fizz Rating (FR)	NP > 30 ppt and FR ≥ 1	Significant source of alkalinity
Potential Ratio (PR)	< 1	Likely to produce acidity
	1 – 2	May produce acidity or alkalinity
	> 2	Likely to produce alkalinity
Net Neutralization Potential (NNP)	< 0 ppt	Likely to produce acidity
	0 – 20 ppt	May produce acidity or alkalinity
	20 – 30 ppt	Likely to produce alkalinity
	> 30 ppt	Very likely to produce alkalinity

- Notes 1. (ppt) = parts per thousand
 2. Reference: Brady, et. al., 1994

10.8 ACID-BASE ACCOUNTING (ABA)

10.8.1 ABA Calculations

The premise of acid-base accounting is that the ability for material at a given location to produce excessive acid or alkaline drainage can be predicted by determining the total amount of acidity and alkalinity that the various rock layers have the potential to produce. Calculations for the accounting process are managed well with a spreadsheet.

For a more thorough description of ABA procedures, refer to [Coal Mine Drainage Prediction and Pollution Prevention in Pennsylvania \(PA-DEP\)](#) and [Evaluation of Acid-Base Accounting Using Computer Spreadsheets \(PA-DEP\)](#).

10.8.2 Worked Examples for Borehole ABA and Alkaline Addition

When evaluating the possible treatment requirements of the site materials, it is necessary to anticipate how the materials can be excavated. In particular, excavations that must be blasted using pre-split methods are limited to lift heights of 30 feet in accordance with Publication 408, Section 203.3. Therefore, to assure constructability and reasonable material management, the ABA assessment should consider material in increments (excavation lift thicknesses) no greater than 30 feet.

For the examples shown in this section, NP_w represents the Neutralization Potential for each sample increment within a managed zone. NP_w is in units of parts per thousand (ppt) $CaCO_3$ equivalent, and is determined by multiplying the NP of a sample by the proportional fraction of the total volume represented by the sample. The fraction of the total volume represented by a sample is often the sample increment thickness (typically 1 to 3 ft.), divided by the total thickness of the lift (30 ft. maximum). This may not always be the case and is dependent upon the geometry of the excavation and stratigraphy. Summing the individual NP_w values for the excavated lift thickness yields the NP of the lift.

$$NP_w = \frac{(NP \text{ of Sample})(Increment \text{ Thickness})}{Lift \text{ Thickness}}$$

In a similar manner, MPA_w is calculated for each sample increment, and the MPA is then determined for the entire lift thickness in accordance with the following:

$$MPA_w = \frac{(MPA \text{ of Sample})(Increment \text{ Thickness})}{Lift \text{ Thickness}}$$

For simplification, the borehole ABA examples shown assume flat-bedded geology (minimal dip), with the horizontal extent of each increment being equal. This will not usually be the case. The actual excavation geometry and depositional geometry of the rock strata need to be considered when calculating excavation volumes. Each project will differ.

The chemistry of adjacent borings should also be compared carefully. When ABA chemistry of adjacent holes differs, judgment should be applied to extrapolate horizontal trends. In some cases, additional testing may be needed if initial test data is not sufficiently conclusive.

For the alkaline addition examples, the rock material and supplemental alkaline material is assumed to be well-blended.

IMPORTANT NOTES about the following Worked Examples:

Borehole ABA Examples:

It should be understood that each of the Borehole ABA examples shown in this section calculate the ABA for only one bore hole. A complete site ABA would involve multiple borings, with the appropriate volume of excavation assigned to each hole. The geometry of the planned excavation and the original site topography would have to be factored in for a complete volumetric analysis. Sites that involve inclined and folded bedding may require mapping or preparation of sections for a complete and accurate accounting of all materials present. A good identification and description of the rock materials, approximate elevations that the materials are expected to be encountered and approximate thickness of the strata, must be provided. The site exploration boreholes and any other pertinent field data should be obtained by a survey crew and mapped. Accurate position and vertical elevation of the boreholes is essential for accurate strata correlation.

Alkaline Addition Examples:

These examples show the general steps for determining the total volume of supplemental alkaline material needed based on the fill chemistry, fill volume, and purity of the supplemental alkaline source. These examples assume all excavated materials will be mixed sufficiently to be considered uniform fill. This may not be a reasonable assumption for very large volumes of excavation involving different types of segmented rock deposits that cannot be blended together. In such cases, alkaline addition rates shall be calculated separately for each type of material that is expected to be excavated and handled separately (see Example 6).

The Calcium Carbonate Equivalent (CCE) of alkaline material is directly related to its purity. Pure calcite, (CaCO₃) has a CCE value of 100% whereas pure dolomite (CaCO₃ MgCO₃) has a CCE of 108%. The CCE of commonly available alkaline materials are shown in [Table 10.8.2-1](#):

Table 10.8.2-1 – Chemical Composition and CCE of Certain Alkaline Material Types¹

Alkaline Material Type	Calcium Carbonate Equivalent
Calcitic Lime - CaCO ₃	85 – 100%
Dolomitic Lime - (CaCO ₃)(MgCO ₃)	85 – 108%
Quick or Burned Lime - (CaO)	150 – 175%
Slaked or Hydrated Lime - Ca(OH) ₂	110 – 135%
Pelletized Lime – CaCO ₃	85 – 100%

Note: 1. Source: Virginia Cooperative Extension - Agronomy Handbook

The following worked examples assume calcitic lime CaCO₃ with a CCE value of 85%. Other forms of alkaline material can be used based on cost and local availability for neutralization. Unless the specific purity of the alkaline material is known and verified, use the lower limit listed for the CaCO₃ equivalent being used. Contract provision must specify the CaCO₃ equivalency value used in determining alkaline addition rates. The contractor must recalculate alkaline addition rates if materials with lower CaCO₃ equivalency values are used during construction.

Worked Example 1 – Borehole ABA

This example includes both high-NP and high-MPA in discrete depositional layers.

Boring R-12, Depth (ft.)	Thickness (ft.)	Fizz	FR	Total Sulfur (%)	NP (ppt)	MPA (ppt)	PR	NNP (ppt)
0.0 - 1.5	1.5	-	-	-	-	-	-	-
1.5 - 4.5	3.0	N	0	0.08	3.5	2.5	1.4	1.0
4.5 - 7.5	3.0	N	0	0.06	5.3	1.9	2.8	3.4
7.5 - 9.0	1.5	SL	1	0.08	17.0	2.5	6.8	14.5
9.0 - 12.0	3.0	N	0	0.06	4.6	1.9	2.5	2.7
12.0 - 15.0	3.0	SL	1	0.04	43.0	1.3	34.4	41.8
15.0 - 18.0	3.0	M	2	0.04	71.3	1.3	57.0	70.1
18.0 - 20.0	2.0	N	0	0.04	12.4	1.3	9.9	11.2
20.0 - 23.0	3.0	ST	3	0.04	98.2	1.3	78.6	97.0
23.0 - 26.0	3.0	M	2	0.02	42.2	0.6	67.5	41.6
26.0 - 28.0	2.0	N	0	0.08	13.7	2.5	5.5	11.2
28.0 - 30.0	2.0	N	0	0.12	5.4	3.8	1.4	1.7
30.0 - 32.0	2.0	N	0	0.04	19.5	1.3	15.6	18.3
32.0 - 34.0	2.0	N	0	0.12	5.5	3.8	1.5	1.8
34.0 - 36.0	2.0	N	0	0.20	8.0	6.3	1.3	1.8
36.0 - 39.0	3.0	N	0	0.03	7.2	0.9	7.7	6.3
39.0 - 42.0	3.0	N	0	1.23	3.4	38.4	0.1	-35.0
42.0 - 45.0	3.0	N	0	2.20	2.2	68.8	0.0	-66.6
45.0 - 47.0	2.0	N	0	2.54	1.5	79.4	0.0	-77.9
47.0 - 49.0	2.0	N	0	2.75	3.5	85.9	0.0	-82.4
49.0 - 51.0	2.0	N	0	0.35	3.5	10.9	0.3	-7.4
51.0 - 53.0	2.0	N	0	1.98	3.5	61.9	0.1	-58.4
53.0 - 55.0	2.0	N	0	0.09	11.2	2.8	4.0	8.4

Figure 10.8.2-1 – Borehole ABA (Example 1)

In this example, the drill column contains areas of both high alkaline potential (shaded blue) and high potential acidity (shaded orange). Because the zone of high potential acidity is isolated (39.0 to 53.0 feet) this scenario is a good candidate for segregated excavation and handling of that material. When this material is excavated, it must be mitigated in some manner, to prevent the production of acid rock drainage. The mitigation may be accomplished by treating the excavated ABR with a supplemental high-alkaline material. As an added measure, the treated rock may also then be encapsulated “high and dry” in an on-site fill, if possible. Mitigation options are discussed in [Section 10.10](#) in greater detail.

See the ABA calculations on the next page for this scenario. [Figure 10.8.2-2](#) shows the result of calculating ABA corresponding to a series of anticipated excavation lifts. The top lift has high-NP layers and the bottom lift has high-MPA layers.

Lift 1	Boring R-12, Depth (ft.)	Thickness (ft.)	Fizz	FR	Total Sulfur (%)	NP (ppt)	MPA (ppt)	PR	NNP (ppt)	volume % (*)	NP _w (ppt) (**)	MPA _w (ppt) (***)
	1.5 – 4.5	3.0	N	0	0.08	3.5	2.5	1.4	1.0	10.5%	0.37	0.26
	4.5 – 7.5	3.0	N	0	0.06	5.3	1.9	2.8	3.4	10.5%	0.56	0.20
	7.5 – 9.0	1.5	SL	1	0.08	17.0	2.5	6.8	14.5	5.3%	0.89	0.13
	9.0 – 12.0	3.0	N	0	0.06	4.6	1.9	2.5	2.7	10.5%	0.48	0.20
	12.0 – 15.0	3.0	SL	1	0.04	43.0	1.3	34.4	41.8	10.5%	4.53	0.13
	15.0 – 18.0	3.0	M	2	0.04	71.3	1.3	57.0	70.1	10.5%	7.51	0.13
	18.0 - 20.0	2.0	N	0	0.04	12.4	1.3	9.9	11.2	7.0%	0.87	0.09
	20.0 – 23.0	3.0	ST	3	0.04	98.2	1.3	78.6	97.0	10.5%	10.34	0.13
	23.0 – 26.0	3.0	M	2	0.02	42.2	0.6	67.5	41.6	10.5%	4.44	0.07
	26.0 – 28.0	2.0	N	0	0.08	13.7	2.5	5.5	11.2	7.0%	0.96	0.18
	28.0 – 30.0	2.0	N	0	0.12	5.4	3.8	1.4	1.7	7.0%	0.38	0.26
Sums =										100%	31.33	1.78
NNP of Lift 1 = NP _w - MPA _w =											29.6 ppt	
PR of Lift 1 = NP _w / MPA _w =											17.6	
Lift 2	Boring R-12, Depth (ft.)	Thickness (ft.)	Fizz	FR	Total Sulfur (%)	NP (ppt)	MPA (ppt)	PR	NNP (ppt)	volume % (*)	NP _w (ppt) (**)	MPA _w (ppt) (***)
	30.0 – 32.0	2.0	N	0	0.04	19.5	1.3	15.6	18.3	8.0%	1.56	0.10
	32.0 – 34.0	2.0	N	0	0.12	5.5	3.8	1.5	1.8	8.0%	0.44	0.30
	34.0 – 36.0	2.0	N	0	0.20	8.0	6.3	1.3	1.8	8.0%	0.64	0.50
	36.0 – 39.0	3.0	N	0	0.03	7.2	0.9	7.7	6.3	12.0%	0.86	0.11
	39.0 – 42.0	3.0	N	0	1.23	3.4	38.4	0.1	-35.0	12.0%	0.41	4.61
	42.0 – 45.0	3.0	N	0	2.20	2.2	68.8	0.0	-66.6	12.0%	0.26	8.25
	45.0 – 47.0	2.0	N	0	2.54	1.5	79.4	0.0	-77.9	8.0%	0.12	6.35
	47.0 – 49.0	2.0	N	0	2.75	3.5	85.9	0.0	-82.4	8.0%	0.28	6.88
	49.0 – 51.0	2.0	N	0	0.35	3.5	10.9	0.3	-7.4	8.0%	0.28	0.88
	51.0 – 53.0	2.0	N	0	1.98	3.5	61.9	0.1	-58.4	8.0%	0.28	4.95
	53.0 – 55.0	2.0	N	0	0.09	11.2	2.8	4.0	8.4	8.0%	0.90	0.23
Sums =										100%	6.03	33.15
NNP of Lift 2 = NP _w - MPA _w =											-27.1 ppt	
PR of Lift 2 = NP _w / MPA _w =											0.2	

* The volumes must sum to 100% for each lift evaluated
 ** Summing the weighted-NP values yields the NP value for the lift
 *** Summing the weighted-MPA values yields the MPA value for the lift

Figure 10.8.2-2 – Lift ABA (Example 1)

Notice that the materials shown in [Figure 10.8.2-2](#) are identical to [Figure 10.8.2-1](#) except that they are now divided into two separate lifts. The lift thicknesses were selected based on: 1)

the notably different chemistry between the top and bottom of the boring and; 2) the anticipated excavation sequence and ability to efficiently handle the material. Lift-1 can therefore be managed separately from the materials deposited below it. Accordingly, separate ABA calculations are completed for each lift. Lift-1 (1.5 - 30 ft.) NNP is calculated to be 29.6 ppt CaCO₃. This material, as a whole, is net-alkaline and should require no supplemental alkaline material. Lift-2 (30 -55 ft.) has a calculated NNP value of -27.1 ppt CaCO₃ and will most certainly require supplemental alkaline material.

The top 1.5 ft. is topsoil to be used elsewhere on the project and is not to be mixed with the fill; therefore, it is excluded from the calculations.

Worked Example 2 – Borehole ABA

This example looks at a column with inter-bedded layers of high-MPA and high-NP materials, and then considers the necessary alkaline addition.

Boring R-13, Depth (ft.)	Thickness (ft.)	Fizz	FR	Total Sulfur (%)	NP (ppt)	MPA (ppt)	PR	NNP (ppt)
0.0 - 1.5	1.5	-	-	-	-	-	-	-
1.5 - 4.5	3.0	N	0	0.08	3.5	2.5	1.4	1.0
4.5 - 7.5	3.0	N	0	1.23	3.4	38.4	0.1	-35.0
7.5 - 9.0	1.5	SL	1	0.08	17.0	2.5	6.8	14.5
9.0 - 12.0	3.0	ST	3	0.04	98.2	1.3	78.6	97.0
12.0 - 15.0	3.0	N	0	0.06	5.3	1.9	2.8	3.4
15.0 - 18.0	3.0	N	0	0.06	4.6	1.9	2.5	2.7
18.0 - 20.0	2.0	N	0	0.04	12.4	1.3	9.9	11.2
20.0 - 23.0	3.0	N	0	2.20	2.2	68.8	0.0	-66.6
23.0 - 26.0	3.0	M	2	0.02	42.2	0.6	67.5	41.6
26.0 - 28.0	2.0	N	0	0.08	13.7	2.5	5.5	11.2
28.0 - 30.0	2.0	N	0	0.12	5.4	3.8	1.4	1.7
30.0 - 32.0	2.0	N	0	0.04	19.5	1.3	15.6	18.3
32.0 - 35.0	3.0	M	2	0.04	71.3	1.3	57.0	70.1
35.0 - 38.0	3.0	N	0	0.03	7.2	0.9	7.7	6.3
38.0 - 40.0	2.0	N	0	2.54	1.5	79.4	0.0	-77.9
40.0 - 42.0	2.0	N	0	0.20	8.0	6.3	1.3	1.8
42.0 - 44.0	2.0	N	0	2.75	3.5	85.9	0.0	-82.4
44.0 - 46.0	2.0	N	0	0.12	5.5	3.8	1.5	1.8
46.0 - 49.0	3.0	SL	1	0.04	43.0	1.3	34.4	41.8
49.0 - 51.0	2.0	N	0	0.35	3.5	10.9	0.3	-7.4
51.0 - 53.0	2.0	N	0	1.98	3.5	61.9	0.1	-58.4
53.0 - 55.0	2.0	N	0	0.09	11.2	2.8	4.0	8.4

Figure 10.8.2-3 – Borehole ABA (Example 2)

In Example 2, the drill column contains increments of both high-alkaline potential (shaded blue) and high-acid potential (shaded orange). The increments are identical to those

shown in Example 1 except for the vertical sequence in which they occur. Because the zones of high potential acidity are now inter-bedded throughout the column in this scenario, this would not be a good candidate for segregated excavation and handling of specific acidic zones. If the buffering from the inter-bedded alkaline zones is not sufficient, the entire volume of material must be mitigated to prevent the production of ARD.

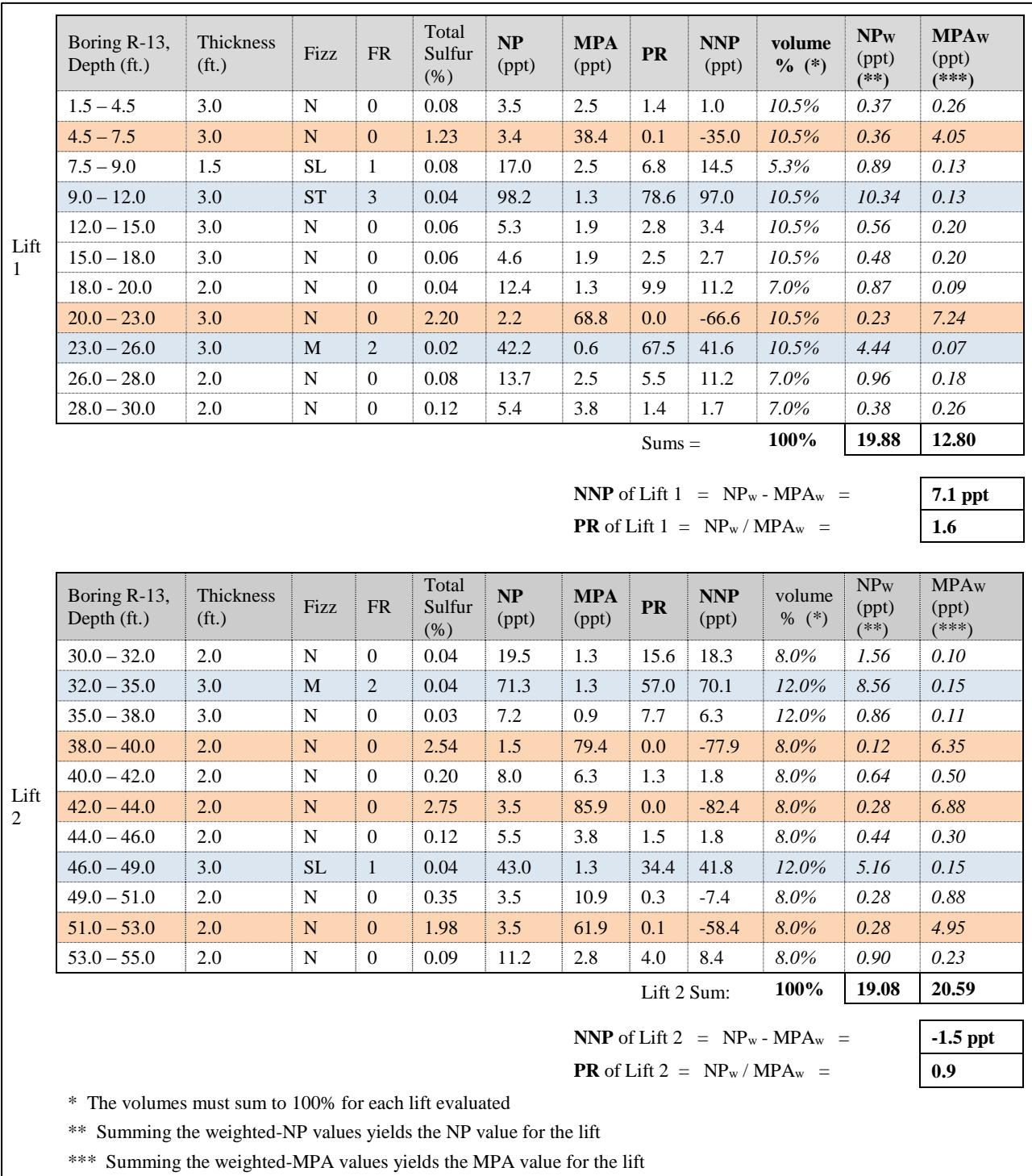


Figure 10.8.2-4 – Lift ABA (Example 2)

[Figure 10.8.2-4](#) shows the result of calculating ABA corresponding to a series of anticipated excavation lifts. Lift-1 is slightly net-alkaline and may require some SAM to provide buffering. Lift-2 is clearly net-acidic with NNP = -1.5 ppt and PR = 0.9. Separate SAM addition rates should be calculated for both lifts.

Worked Example 3 – Borehole ABA

This is an example where the ABA results are less definitive relative to whether ABR is a problem, and whether treatment is required.

Boring R-14, Depth (ft.)	Thickness (ft.)	Fizz	FR	Total Sulfur (%)	NP (ppt)	MPA (ppt)	PR	NNP (ppt)
0.0 - 1.0	1.0	N	0	0.00	0.28	0.00	-0.51	0.28
1.0 - 4.0	3.0	N	0	0.00	0.31	0.00	0.49	0.31
4.0 - 7.0	3.0	N	0	0.01	-0.20	0.31	-1.94	-0.51
7.0 - 10.0	3.0	N	0	0.01	0.30	0.31	0.87	-0.01
10.0 - 13.0	3.0	N	0	0.01	0.62	0.31	1.87	0.31
13.0 - 16.0	3.0	N	0	0.01	0.72	0.31	1.55	0.41
16.0 - 19.0	3.0	N	0	0.01	0.59	0.31	1.10	0.28
19.0 - 22.0	3.0	N	0	0.02	0.88	0.63	0.33	0.26
22.0 - 25.0	3.0	N	0	0.03	-0.22	0.94	-1.05	-1.16
25.0 - 28.0	3.0	N	0	0.02	0.56	0.63	0.57	-0.06
28.0 - 31.0	3.0	N	0	0.02	0.18	0.63	-0.22	-0.45
31.0 - 34.0	3.0	N	0	0.01	-0.12	0.31	-0.23	-0.43
34.0 - 37.0	3.0	N	0	0.00	0.45	0.00	0.33	0.45
37.0 - 40.0	3.0	N	0	0.00	1.09	0.00	0.29	1.09

Figure 10.8.2-5 – Borehole ABA (Example 3)

In this example, there is no single layer that is anticipated as a significant acid producer since the total sulfur ranges only from 0.00 to 0.03 percent. And while there are several negative PR's, NP's and NNP's, and no significant sources of alkaline, the MPA is very low. While some values are in ranges indicating a potential for ABR (relative to values indicated in [Table 10.7.4-1](#)), there is also little source for acidity. Situations of this nature must be scrutinized closely, to assess the need for treatment or mitigation.

[Figure 10.8.2-6](#) shows the result of calculating ABA corresponding to a series of anticipated excavation lifts. Both Lift-1 and Lift-2 are essentially acid-base neutral. Neither has elevated sulfur contents. Situations such as this would not typically require alkaline addition unless other factors, such as past experience in similar geology, have proven otherwise. Rock type and color can also help predict the acid-base chemistry in these cases.

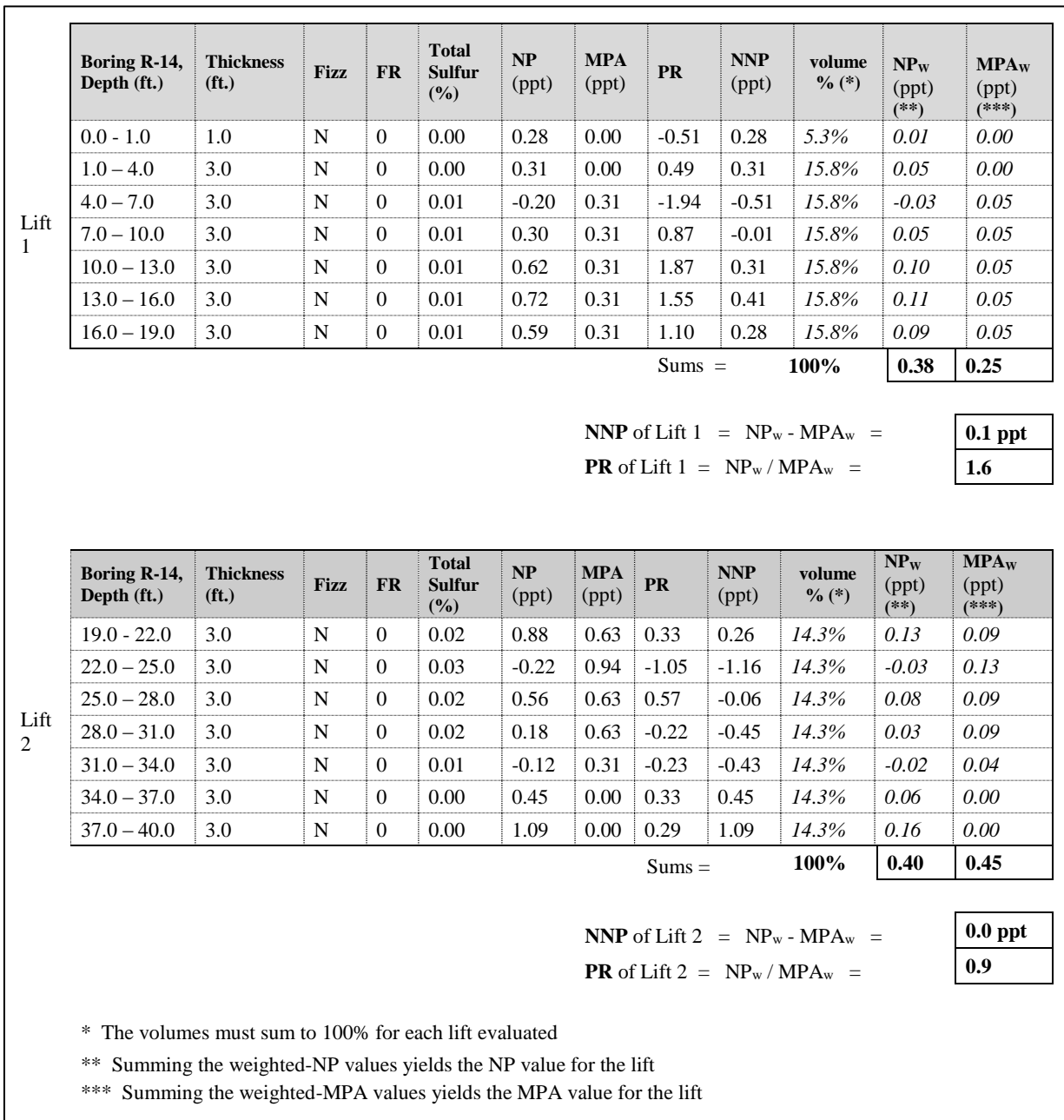


Figure 10.8.2-6 – Lift ABA (Example 3)

Worked Example 4 – Alkaline Addition for Uniform Rock Deposit

This calculation anticipates all materials, including Supplemental Alkaline Material (SAM), can be adequately blended together.

This example determines the total quantity of SAM needed for neutralization of ABR waste material involving only one, uniform rock deposit. This scenario is likely to occur with a smaller volume, shallow excavation such as a structure foundation.

Step 1: Tabulate the laboratory testing results:

Site Material	Quantity to be Excavated	NP (ppt CaCO ₃)	% Sulfur	pH	Fizz Rating
Carbonaceous Black Shale	1,000 tons	0.34	0.55%	5.8	0

Step 2: Calculate NNP of the ABR:

$$\begin{aligned} \text{MPA}_{\text{Shale}} &= (0.55)(31.25) \\ &= \underline{17.2 \text{ ppt CaCO}_3} \end{aligned}$$

$$\begin{aligned} \text{NNP}_{\text{Shale}} &= \text{NP}_{\text{Shale}} - \text{MPA}_{\text{Shale}} \\ &= 0.34 - 17.19 \\ &= \underline{-16.85 \text{ ppt CaCO}_3} \quad (<0, \text{ therefore, likely to be acidic}) \end{aligned}$$

Step 3: Calculate PR of the ABR:

$$\text{PR}_{\text{Shale}} = \text{NP}/\text{MPA} = 0.34/17.19 = \underline{0.02} \quad (<1, \text{ therefore, likely to be acidic})$$

Step 4: Calculate required alkaline addition:

To help assure ARD will not be generated, set the target site NNP equal to a minimum of 12.0 ppt CaCO₃. A higher target NNP can be selected based on local experience or regulatory guidance.

$$\begin{aligned} \text{NNP}_{\text{Deficiency}} &= (\text{NNP}_{\text{Target}}) - (\text{NNP}_{\text{Shale}}) \\ &= (12.0) - (-16.85) \\ &= \underline{28.85 \text{ ppt CaCO}_3} \end{aligned}$$

Apply a factor-of-safety (FS = 2.0) to assure adequate alkalinity is available. This helps to offset the effect of imperfect mixing.

$$\begin{aligned} \text{NNP}_{\text{Required}} &= (\text{NNP}_{\text{Deficiency}}) \times \text{FS} \\ &= (28.85) \times (2.0) \\ &= \underline{57.7 \text{ ppt CaCO}_3} \quad \text{or} \quad \underline{57.7 \text{ tons of pure lime} / 1000 \text{ tons of rock}} \end{aligned}$$

Step 5: Determine the quantity of the SAM selected for the project.

This is a function of the purity of the imported alkaline material. In this case, an 85% CaCO₃ equivalent (CCE) material (NP = 850, MPA = 0) is selected:

$$\begin{aligned} \text{Total SAM required for the project} &= (\text{NNP}_{\text{Required}}) \times (\text{Overburden mass}) / (\text{CCE}) \\ &= (57.7 \text{ tons}/1000 \text{ tons}) \times (1,000 \text{ tons}) / (0.85) \\ &= \underline{68 \text{ tons}} \end{aligned}$$

Worked Example 5 – Alkaline Addition for Deposits with Multiple Rock Types

This calculation anticipates all materials (including SAM) can be adequately blended together.

This example determines the total quantity of SAM needed for neutralization of ABR waste material involving deposits with more than one rock type which can be adequately blended during excavation and placement. This scenario could occur when excavating deposits of changing or multiple lithologies, or when inter-bedded or steeply-dipping deposits of different lithologies are excavated.

Step 1: Tabulate site overburden quantity and laboratory testing results:

Site Material	Quantity to be Excavated	NP (ppt CaCO ₃)	% Sulfur	pH	Fizz Rating
Shale	20,000 tons (40% of site volume)	0.6	0.52%	6.5	0
Sandstone	30,000 tons (60% of site volume)	5.0	0.02%	7.0	0

Step 2: Calculate NNP of Site:

$$\begin{aligned}
 NP_{\text{Site}} &= (\% \text{ Shale}) NP_{\text{Shale}} + (\% \text{ SS}) NP_{\text{SS}} \\
 &= (0.40)(0.6) + (0.60)(5.0) \\
 &= 0.24 + 3.0 = \underline{3.24 \text{ ppt CaCO}_3}
 \end{aligned}$$

$$\begin{aligned}
 MPA_{\text{Site}} &= (\% \text{ Shale}) MPA_{\text{Shale}} + (\% \text{ SS}) MPA_{\text{SS}} \\
 &= (0.40)(0.52)(31.25) + (0.60)(0.02)(31.25) \\
 &= 6.5 + 0.38 = \underline{6.88 \text{ ppt CaCO}_3}
 \end{aligned}$$

$$\begin{aligned}
 NNP_{\text{Site}} &= NP_{\text{Site}} - MPA_{\text{Site}} \\
 &= 3.24 - 6.88 = \underline{-3.64 \text{ ppt CaCO}_3} \quad (<0, \text{ therefore, likely to be acidic})
 \end{aligned}$$

Step 3: Calculate PR of the ABR:

$$PR_{\text{Site}} = NP/MPA = 3.24/6.88 = \underline{0.47} \quad (<1, \text{ therefore, likely to be acidic})$$

Step 4: Calculate required alkaline addition:

To help assure ARD will not be generated, set the target site NNP equal to a minimum of 12.0 ppt CaCO₃. A higher target NNP can be selected based on local experience or regulatory guidance.

$$\begin{aligned}
 NNP_{\text{Deficiency}} &= (NNP_{\text{Target}}) - (NNP_{\text{Site}}) \\
 &= (12.0) - (-3.64) = \underline{15.64 \text{ ppt CaCO}_3}
 \end{aligned}$$

Apply a factor-of-safety (FS = 2.0) to assure adequate alkalinity is available. This helps to offset the effect of imperfect mixing.

$$\begin{aligned} \text{NNP}_{\text{Required}} &= (\text{NNP}_{\text{Deficiency}}) \times \text{FS} \\ &= (15.64) \times (2.0) = \underline{31.28 \text{ ppt CaCO}_3} \text{ or } \underline{31.28 \text{ tons lime} / 1000 \text{ tons fill}} \end{aligned}$$

Step 5: Determine the quantity of the SAM selected for the project:

This is a function of the purity of the imported alkaline material. In this case, an 85% CaCO₃ equivalent (CCE) material (NP = 850, MPA = 0) is selected:

$$\begin{aligned} \text{Total SAM required for the project} &= (\text{NNP}_{\text{Required}}) \times (\text{Overburden mass}) / (\text{CCE}) \\ &= (31.28 \text{ tons}/1000 \text{ tons}) \times (50,000 \text{ tons}) / (0.85) \\ &= \underline{1,840 \text{ tons}} \end{aligned}$$

Worked Example 6 – Alkaline Addition for Multiple Rock Deposits (Segregated)

This calculation anticipates conditions when excavations must be zoned (e.g. when encountering isolated zones of high acid potential) and deposits will not be blended together, but managed separately.

This example determines the total quantities of SAM needed for neutralization of ABR waste material where excavations must be zoned, having deposits of substantially different acid potential, which must be excavated, treated and placed separately.

Step 1: Tabulate site overburden quantity and laboratory testing results:

Site Material	Deposit Thickness	Quantity to be Excavated & Treated	NP (ppt CaCO ₃)	% Sulfur	pH	Fizz Rating
Black Shale	15 ft.	15,000 tons (25% of site volume)	10.0	0.56%	6.0	0
Coal*	6 ft.	5,000 tons (<1% of site volume, include only 10% 'loss' volume in SAM calculations)	0.0	2.2%	5.0	0
Gray Siltstone	45 ft.	45,000 tons (75% of site volume)	12.0	0.11%	6.5	0

***Note:** In this example the coal deposit is sufficient thickness such that the coal should be excavated separately and hauled to a coal facility for disposal or marketing, and it should not be blended with the other fill materials. It is not possible to remove 100% of the coal deposit. Assume 90% of the material can be removed, i.e., 10% loss. If the very high sulfur content material were not coal, or it is not practical to mine or separate the coal, appropriate calculations for SAM would be conducted, and the material would be excavated, treated and placed in a

separate operation from the overlying shale and underlying siltstone materials, to the extent possible.

Step 2: Calculate NNP of Each Rock Type:

$$\begin{aligned} \text{MPA}_{\text{Shale}} &= \% \text{ Sulfur} \times 31.25 \\ &= (0.56)(31.25) \\ &= \underline{17.5 \text{ ppt CaCO}_3} \end{aligned}$$

$$\begin{aligned} \text{NNP}_{\text{Shale}} &= \text{NP}_{\text{Shale}} - \text{MPA}_{\text{Shale}} \\ &= 10.0 - 17.5 \\ &= \underline{-7.5 \text{ ppt CaCO}_3} \quad (<0, \text{ likely to be acidic}) \end{aligned}$$

$$\begin{aligned} \text{MPA}_{\text{Coal}} &= \% \text{ Sulfur} \times 31.25 \\ &= (2.2)(31.25) \\ &= \underline{68.75 \text{ ppt CaCO}_3} \end{aligned}$$

$$\begin{aligned} \text{NNP}_{\text{Coal}} &= \text{NP}_{\text{Silt}} - \text{MPA}_{\text{Silt}} \\ &= 0 - 68.75 \\ &= \underline{-68.75 \text{ ppt CaCO}_3} \quad (<<0, \text{ acidic}) \end{aligned}$$

$$\begin{aligned} \text{MPA}_{\text{Silt}} &= \% \text{ Sulfur} \times 31.25 \\ &= (0.11)(31.25) \\ &= \underline{3.44 \text{ ppt CaCO}_3} \end{aligned}$$

$$\begin{aligned} \text{NNP}_{\text{Silt}} &= \text{NP}_{\text{Silt}} - \text{MPA}_{\text{Silt}} \\ &= 12.0 - 3.44 \\ &= \underline{8.56 \text{ ppt CaCO}_3} \quad (<20, \text{ possibly acidic}) \end{aligned}$$

Step 3: Calculate required alkaline addition for the Shale:

To help assure ARD will not be generated, set the target site NNP equal to a minimum of 12.0 ppt CaCO₃. A higher target NNP can be selected based on local experience or regulatory guidance.

$$\begin{aligned} \text{Shale NNP}_{\text{Deficiency}} &= (\text{NNP}_{\text{Target}}) - (\text{NNP}_{\text{Shale}}) \\ &= (12.0) - (-7.5) \\ &= \underline{19.5 \text{ ppt CaCO}_3} \end{aligned}$$

Apply a factor-of-safety (FS = 2.0) to assure adequate alkalinity is available. This helps to offset the effect of imperfect mixing.

$$\begin{aligned} \text{Shale NNP}_{\text{Required}} &= (\text{NNP}_{\text{Deficiency}}) \times \text{FS} \\ &= (19.5) \times (2.0) \\ &= \underline{39.0 \text{ ppt CaCO}_3} \quad \text{or} \quad \underline{39.0 \text{ tons lime} / 1000 \text{ tons fill}} \end{aligned}$$

Step 4: Determine the quantity of the specific SAM need for the Shale:

This is a function of rock volume and the purity of the imported alkaline material. In this case, an 85% CaCO₃ equivalent (CCE) material (NP = 850, MPA = 0) is selected:

$$\begin{aligned} \text{Total SAM required for the Shale} &= (\text{NNP}_{\text{Required}}) \times (\text{Shale mass}) / (\text{CCE}) \\ &= (39.0 \text{ tons}/1000 \text{ tons}) \times (15,000 \text{ tons}) / (0.85) \\ &= \underline{688 \text{ tons}} \end{aligned}$$

Step 5: Calculate required alkaline addition for the Coal (loss):

To help assure ARD will not be generated, set the target site NNP equal to a minimum of 12.0 ppt CaCO₃. A higher target NNP can be selected based on local experience or regulatory guidance.

$$\begin{aligned} \text{Coal NNP}_{\text{Deficiency}} &= (\text{NNP}_{\text{Target}}) - (\text{NNP}_{\text{Coal}}) \\ &= (12.0) - (-68.75) \\ &= \underline{80.75 \text{ ppt CaCO}_3} \end{aligned}$$

Apply a factor-of-safety (FS = 2.0) to assure adequate alkalinity is available. This helps to offset the effect of imperfect mixing.

$$\begin{aligned} \text{Coal NNP}_{\text{Required}} &= (\text{NNP}_{\text{Deficiency}}) \times \text{FS} \\ &= (80.75) \times (2.0) \\ &= \underline{161.5 \text{ ppt CaCO}_3} \text{ or } \underline{161.5 \text{ tons lime} / 1000 \text{ tons fill}} \end{aligned}$$

Step 6: Determine the quantity of the specific SAM need for the Coal (loss):

An 85% CaCO₃ equivalent (CCE) material is selected. Ten percent of the coal will remain as loss. In this case the coal loss is 500 tons.

$$\begin{aligned} \text{Total SAM required for the Coal (loss)} &= (\text{NNP}_{\text{Required}}) \times (\text{Coal loss mass}) / (\text{CCE}) \\ &= (161.5 \text{ tons}/1000 \text{ tons}) \times (500 \text{ tons}) / (0.85) \\ &= \underline{81 \text{ tons}} \end{aligned}$$

Step 7: Calculate required SAM for the Siltstone:

To help assure ARD will not be generated, set the target site NNP equal to a minimum of 12.0 ppt CaCO₃. For this example, a target NNP = 20 is selected based on local experience with this rock type.

$$\begin{aligned} \text{Siltstone NNP}_{\text{Deficiency}} &= (\text{NNP}_{\text{Target}}) - (\text{NNP}_{\text{Silt}}) \\ &= (20.0) - (5.86) \\ &= \underline{14.14 \text{ ppt CaCO}_3} \end{aligned}$$

Apply a factor-of-safety (FS = 2.0) to assure adequate alkalinity is available. This helps to offset the effect of imperfect mixing.

$$\begin{aligned} \text{Siltstone NNP}_{\text{Required}} &= (\text{NNP}_{\text{Deficiency}}) \times \text{FS} \\ &= (14.14) \times (2.0) \\ &= \underline{28.3 \text{ ppt CaCO}_3} \quad \text{or} \quad \underline{28.3 \text{ tons lime} / 1000 \text{ tons fill}} \end{aligned}$$

Step 8: Determine the quantity of the specific SAM needed for the Siltstone:

Imported alkaline material with a 85% CaCO₃ equivalent (CCE) is selected:

$$\begin{aligned} \text{Total SAM required for the Siltstone} &= (\text{NNP}_{\text{Required}}) \times (\text{Siltstone mass}) / (\text{CCE}) \\ &= (28.3 \text{ tons}/1000 \text{ tons}) \times (45,000 \text{ tons}) / (0.85) \\ &= \underline{1,499 \text{ tons}} \end{aligned}$$

Step 9: Determine the total quantity of 85% pure SAM needed for the project:

$$\text{SAM}_{\text{Shale}} + \text{SAM}_{\text{Coal}} + \text{SAM}_{\text{Silt}} = 688 + 81 + 1499 = \underline{2,268 \text{ tons}}$$

In the above example, if the middle layer were a 12-foot thickness of high-acid potential rock, overlain by a five-foot thickness of high-alkaline potential rock, and underlain by a three-foot thickness of high-alkaline potential rock, and the materials can be excavated during the same blasting shot, then the three materials can be included in a single acid-base accounting calculation to determine the required SAM. The high-alkaline rock materials can serve to reduce the required SAM, so long as the materials are thoroughly mixed during placement, and the anticipated particle sizes of the shot materials is taken into account.

10.9 ACID NEUTRALIZATION METHODS

The goal of any excavation in ABR is to avoid the generation of ARD and if proper mitigation techniques have been followed, the production of ARD is in all probability minimized. In some cases, due to site specific and or environmental constraints, generation of ARD is unavoidable and must be treated to the level of control required. The intent of this guidance document is not to be a design manual for the treatment of ARD. If a site requires water treatment, a qualified professional engineer should evaluate the site water conditions and design an appropriate treatment system. In such cases, PA-DEP must be consulted during the project design phase. The need for any required permits must also be determined, and these must be obtained from PA-DEP as necessary. Document all consultations with PA-DEP and obtain all recommendations in writing. The project design manager should maintain a technical file that contains written documentation of all correspondence, consultations, recommendations and requirements from the PA-DEP.

10.9.1 Active Treatment

Treatment systems are classified as active or passive. The type of system required will depend upon the chemistry of the ARD. Active treatment systems are typically more labor and

maintenance intensive as these systems involve mixing of chemicals, pumps, tanks, and filtration to function as designed. Often these systems generate byproducts which require disposal on a regular basis. Active treatment systems are typically used to treat higher levels of acidity and dissolved metals with sodium hydroxide (NaOH). Active treatments may be required in the early stages of oxidation when acid production rates are highest, while passive treatment may be sufficient as the acid production rate slows with the oxidation of the surface rock.

10.9.2 Passive Treatment

Advantages of this system are: less labor intensive, generally require only occasional maintenance; do not require use of hazardous chemicals; is effective in removal of metals; and takes advantage of naturally occurring chemical and biological processes. Types of passive treatment systems include anoxic limestone drains, open limestone channels, limestone settling ponds, and wetlands. Disadvantages include: does not work well for large flows (> 500 gpm); large treatment area needed; high construction costs; and highly acid water (pH <3) can be difficult to treat. The type of passive system(s) selected is based on the water chemistry and flow at the project site. The following subsections briefly describe some of the types of passive treatment systems available. For detailed guidance for passive treatment design, refer to the PA-DEP Engineering Manual for Surface Mining Operations, Technical Guidance Document 563-0300-010.

10.9.2.1 Wetlands

The way in which a wetland is constructed ultimately affects how acidic water treatment occurs. Two construction types are: 1) "aerobic" wetlands consisting of vegetation planted in shallow (<12 inches), relatively impermeable sediments comprised of soil, clay or mine spoil; and 2) "anaerobic" wetlands consisting of vegetation planted into deep (>12 inches), permeable sediments comprised of soil, peat moss, spent mushroom compost, sawdust, straw/manure, hay bales, or a variety of other organic mixtures which are underlain or admixed with limestone.

Aerobic wetlands promote metal oxidation and hydrolysis, thereby causing precipitation and physical retention of Fe, Al, and Mn oxyhydroxides. Successful metal removal depends on dissolved metal concentrations, dissolved oxygen content, pH and net acidity of the mine water, the presence of active microbial biomass, and detention time of the water in the wetland.

Anaerobic wetlands promote metal oxidation and hydrolysis in aerobic surface layers, but primarily rely on chemical and microbial reduction reactions to precipitate metals and neutralize acidity. The water infiltrates through thick permeable organic subsurface sediment that becomes anaerobic due to high biological oxygen demand.

10.9.2.2 Limestone Settling Ponds

Water is collected and retained to allow suspended solids sufficient time to precipitate or "drop out" of water. Iron settles to the bottom of the pond and collects there which after time need to be removed. For ARD high in dissolved iron, a settling pond's large surface area permits

atmospheric oxygen to dissolve into the acidic water which reacts with the iron to form iron oxide (yellowboy), which subsequently settles out.

10.9.2.3 Anoxic Limestone Drains

Anoxic Limestone Drains (ALDs) are buried cells or trenches of limestone into which anoxic water is introduced. The limestone dissolves in the acid water, raises pH, and adds alkalinity. Under anoxic conditions, the limestone does not coat or armor with Fe hydroxides because Fe^{+2} does not precipitate as $Fe(OH)_2$ at pH 6.0.

10.9.2.4 Open Limestone Channels

Open Limestone Channels (OLCs) are another means of introducing alkalinity to acid water. Long channels of limestone can be used to convey acid water to a stream or other discharge point. Based on flows and acidity concentrations, cross sections of stream channels (widths and heights) can be designed with calculated amounts of limestone (which will become armored) to treat the water. OLCs work best where the channel is constructed on slopes steeper than 5H:1V and where flow velocities keep metal hydroxides in suspension, thereby limiting oxide accumulation and the subsequent plugging of open voids within the limestone channel lining. Utilizing OLCs with other passive systems can maximize treatment and metal removal.

10.10 MITIGATION

10.10.1 Mitigation Tools

The goal of any treatment or mitigation plan is to prevent the formation of acid – more specifically to prevent the acid generation reaction from starting and getting established. There are three general “tools” available in designing an effective strategy to prevent acid production:

1. Buffering (alkaline addition)
2. Deny moisture
3. Deny oxygen

Each of these methods alone could be adequate to provide protection, however the ability to maintain a consistent moisture-free or oxygen-free condition long term is not readily achievable. For this reason, a more fail-safe strategy involving a practical combination of all three methods should be used, with buffering (alkaline addition) as the primary method used in most cases. Minimizing availability of oxygen and moisture are viewed as additional mitigation strategies. One or more of these tools are incorporated in each of the methods discussed in detail in the following subsections.

10.10.1.1 Alkaline Addition

A straightforward and common approach to preventing acid generation is to add sufficient alkaline to maintain an alkaline environment and neutralize all potential acidity. It is important to note that the primary purpose of alkaline addition (maintaining an alkaline environment) is to inhibit the formation of acid. The high pH of an alkaline environment

interferes with the chemical and biological processes that form acid. Neutralization is in some sense a secondary function, providing a means to counteract and neutralize any acid that is produced.

Common sources of alkalinity include calcite (CaCO_3 also known as calcium carbonate) and dolomite ($\text{CaMg}(\text{CO}_3)_2$), with a less common source being calcium hydroxide ($\text{Ca}(\text{OH})_2$) applied as a liquid slurry. As discussed, in order to assure sufficient buffering, and since the acid reaction occurs at a greater rate than alkaline production, a factor-of-safety must be applied to the alkaline addition. A minimum factor-of-safety of 2.0 must be applied for alkaline addition. This is done by first calculating the alkaline addition rate (lbs. alkalinity/ton fill) to achieve a target NNP greater than zero (a target NNP of 12 to 20 is typically selected), and then multiplying this rate by two. Higher addition rates may be used if justified or required.

The acid potential in rock will likely vary considerably with depth, and therefore specified alkaline-addition rates for proposed rock excavations may have to be adjusted accordingly. Therefore, the results indicated by the acid-base accounting of a given boring column, may not represent the appropriate alkaline addition rate. If a certain zone has significantly higher levels of potential acidity, and the zone can be handled separately during excavation, the appropriate alkaline addition rate should be specified for that zone. The alkaline addition rate for zones with significantly different levels of potential acidity should be calculated and specified as required for that zone. When considering zone or layer specific alkaline addition rates, the practicality of material management during construction operations must also be considered. The required alkaline-addition rates required to address a stratigraphically variable acid potential, need to be balanced with practical construction operations and sequencing. Ultimately, both the constructability and anticipated costs will indicate if a segregated, zoned excavation and variable alkaline-addition rates are a feasible approach.

If alkaline addition costs are high, then closer scrutiny may be necessary in scheduling, sequencing and handling procedures, sacrificing some productivity, or requiring qualified personnel on-site during all excavation operations to identifying those areas requiring higher alkaline rates. If the material to be excavated has a more consistent level of potential acidity, alkaline addition costs are low relative to the cost of impacting production, the volume of material necessary to treat is low, or the cost of qualified on-site personnel is excessive, then a uniform alkaline addition rate may be more appropriate.

The Importance of Particle Size: A factor that must be considered in alkaline addition is the particle size of both the ABR and the added alkaline material. When possible and practical, the particle size of the excavated ABR material should be kept to a maximum. Maximum particle size will provide minimum exposed surface area (available to oxygen and water), which will also help control the rate of acid production. The particle size of the added alkaline must be smaller than that of the suspected ABR. The size of the alkaline particles determines their short-term and long-term “availability” or effectiveness as a source of alkaline.

The neutralization potential of alkaline particles smaller than the No. 60 sieve (250 μm) is considered 100% “available”, that is, effective immediately and within a period of approximately one year. Particles larger than the No. 60 sieve (250 μm) have less surface area

and need years to weather sufficiently so the core of each particle (not just the surface) can react chemically to any surrounding acidic conditions. Alkaline particles larger than 3/8 inch (0.375 in.) are considered to have limited short-term effectiveness due to probable oxidation and armoring of the particle surface. The particles literally form a rust coating that limits the availability of the calcium carbonate beneath.

Small alkaline particles (No. 60 sieve size and smaller) also have a potential disadvantage than needs to be recognized. While providing a readily available source of alkaline, the small particle size also makes the material potentially highly mobile when mixed in a matrix of relatively large particles. This gap-grading allows the fine particles to migrate. Since one of the primary functions of the added alkaline is to maintain an alkaline environment to inhibit acid production, it is important that the added alkaline remain mixed uniformly through the treated mass. The No. 60-sized material would tend to migrate to the bottom of a treated fill, where it would be available and capable of neutralizing any acid solutions that form, but it would not be serving the more important function of inhibiting acid formation, and preventing the acid production cycle from starting.

To prevent the migration of “available” alkaline particles (< No. 60 sieve) through potentially open graded excavated ABR a mix of well-graded limestone aggregate and agricultural lime should be used for the added alkaline material. The proportioning of coarse aggregate limestone and agricultural lime would have to be based on the size and gradation of the excavated ABR. [Table 10.10.1.1-1](#) provides required proportioning based upon excavated material particle size. The proportioning value can be modified as appropriate based upon the actual particle size of excavated ABR. If the actual particle size of the excavated ABR does not match the alkaline particle size distribution assumed during design, the proportioning rate and particle size distribution is adjusted as appropriate.

If the quality of rock to be excavated is anticipated to vary, resulting in variable excavated rock particle size, the proportioning rate must be adjusted during construction as needed. For example, ABR comprised of highly fractured and thinly bedded shale excavated from the top half of a cut and a competent slightly fractured, thickly bedded sandstone excavated from the bottom half of a cut will require different alkaline particle size proportions to limit alkaline migration, but also be available to successfully neutralize any potential acidity. In order to accommodate potential differences between anticipated and actual particle size distribution of excavated ABR, alkaline addition materials, (i.e., 2A limestone aggregate and agricultural lime) should be bid as separate pay items.

Table 10.10.1.1-1 – Particle Size for Alkaline Addition

Characterization of ABR Excavation	Alkaline Migration Potential	Acid Production Potential	Alkaline Addition Mix^{1,2} (Particle Size/Proportions)
Clean Shot Rock <ul style="list-style-type: none"> • Minimal fines content • Large particle size • Relatively uniform particle size • Angular to Blocky fragments • Very competent 	High	Low	75% Limestone 2A 25% Agricultural Lime
Dirty Shot Rock <ul style="list-style-type: none"> • Significant fines • Generally well-graded • Sub-angular to angular fragments • Wide range in competency 	Moderate	Moderate	50% Limestone 2A 50% Agricultural Lime
Highly-Fragmented Rock <ul style="list-style-type: none"> • High fines content • Small particle size • Well-graded to poorly-graded • Variable particle shape • Sub-angular to flat elongated 	Low	High	25% Limestone 2A 75% Agricultural Lime

- Notes: 1. Agricultural Lime must have 100% passing No. 60 sieve
2. Both types of SAM (Limestone 2A and Agricultural Lime) must be thoroughly mixed with the ABR before compaction. SAM is not to be placed and compacted in separate layers or lifts.

As detailed in the worked examples (refer to [Section 10.8](#)), the calculated amount of active CaCO₃ required to neutralize the ABR is doubled by applying a factor-of-safety (FS = 2.0). This helps to assure adequate alkalinity is available. This also helps to offset the effects of imperfect mixing, the lower alkaline availability for the coarse aggregate limestone particles, and the natural variability of rock formations. Added to the calculated factor-of-safety, is a built-in safety by the larger particle size of the ABR, which limits the rate of acid production. Just as the larger particle size of the added alkaline material limits the availability of alkaline due to reduced surface area, the large particle size of ABR also limits potential acid production for the same reason. Much of the acid potential is isolated inside the larger particles, greatly reducing the rate at which the potential acidity can develop.

If site conditions dictate, a higher factor-of-safety should be established to assure adequate alkalinity is available. Therefore, the proportion of alkaline particles (agricultural lime)

comprises 50% or more of the alkaline addition for high to moderate acid production. For clean shot rock, with predominantly large particle sizes, the proportion of agricultural lime is less than 50% due to the lower potential for acid production.

The Importance of Mixing: The added alkaline must be thoroughly mixed with the suspected ABR, to assure proper distribution and alkaline availability. The following method of mixing has proven to be effective and should yield acceptable results for most projects:

1. Determine proper ratio (by weight) of imported alkaline to acid-bearing fill
2. Determine capacity (by weight) of project haul trucks
3. Determine necessary quantity of imported alkaline to be added to each truck load
4. Load haul truck with excavated acid-bearing material
5. Load the pre-determined amount of imported/supplemental alkaline material (SAM) to the loaded truck. It is not necessary to weigh each load. The required quantity can be closely correlated to the volume of the loading bucket.
6. Unload the truck at the designated fill area and blade each load to the required lift thickness. Additional mixing can be achieved by blading, disking, or raking with rock rippers as necessary dependent upon the size of the constituents and visual appearance of the bladed material.

10.10.1.2 Encapsulation (Deny Oxygen and Deny Water)

All fills of ABR or sulfate soils must be encapsulated to isolate the acid-bearing material, as much as practical, from oxygen and water. All practical efforts should be made to keep the acid-bearing material dry until encapsulation is completed. Encapsulation is always performed in combination with alkaline addition. The material must be encapsulated in the fill, placed “high and dry”, a minimum of 5 feet above the 100-year floodplain elevation and the seasonal high groundwater elevation. The encapsulation and high placement will limit exposure of the ABR to both oxygen and water.

In areas where subsurface structures are required (e.g., drainage pipes, inlets, sign foundations, guiderail posts) the capping soil must extend a minimum depth of three feet below any such structure.

If the gradation of the ABR/Alkaline composite fill is significantly different than the foundation soil then a layer of Class-4, Type-A geotextile should be placed prior to placement of the fill to prevent mixing, migration, and loss of alkaline addition material. Once the alkaline addition has been completed, the treated fill is considered encapsulated when a minimum of 3 feet of capping on top, bottom, and sides. The capping must consist of low permeability, fine-grained soil having the following properties:

- **Fines $\geq 35\%$** (min. 35% passing the No. 200 (75 μm) sieve)
- **Plasticity Index ≥ 6** (min. PI = 6 for fraction passing No. 40 (425 μm) sieve)

The soil capping must be underlain by a Class-4, Type-A geotextile to prevent migration and loss of the soil into the rock fill. When the ABR fill is very coarse, angular and/or open-graded, the ABR must be choked off with granular material to prevent puncture of the geotextile.

The geotextile and soil capping on the slopes must be placed concurrently with the ABR fill. The slope of any encapsulated ABR fills is limited to a maximum 2.5H:1V, horizontal to vertical. While 2H:1V slopes are common and acceptable, and 2H:1V is more than adequate for internal stability a fill with a rock core, the risks associated with a surficial slide or erosion of an encapsulated ABR fill dictate a flatter 2.5H:1V slope.

10.10.1.3 Submergence (Deny Oxygen)

An effective option to encapsulation is permanent submergence in water. This too will deny the necessary oxygen for acid production (even though in the presence of ample moisture), provided the Dissolved Oxygen (DO) concentration of the water does not exceed that of uncontaminated regional groundwater, approximately 4.0 mg/L DO.

Situations that normally present a design or construction challenge can sometimes be exploited. For example, with excavations that contain ABR below the water table, keeping the excavation flooded (if practical) will limit the exposure to oxygen, keeping oxidation of ABR under control. This may be an effective approach for open excavations for structure foundation construction near river/stream environments. In such cases, it may be readily possible to keep the excavation submerged until foundation construction is complete, or other protective measures can be implemented.

10.10.2 Additional Considerations for Treatment and Mitigation

While the above “tools” are very effective, the preferred option is to always avoid excavating or exposing ABR when possible and practical, by alignment or grade adjustments. When this is not possible, treatment and mitigation is typically necessary. It is most critical to know what the acid-bearing strata are (i.e. be able to identify the acid-bearing rock or soil) and know where they are located (depth, thickness and lateral extent). For flat-bedded geology, adequate boring data and sample testing may be sufficient. However, in more complex geology (folded and inclined bedding), subsurface mapping and/or cross sections indicating the location of suspected ABR strata, may be required.

The same information should be provided for any high neutralization potential (NP) strata identified during the testing and ABA processes, if in developing the mitigation plan the alkaline material can be used to help buffer the acid-bearing materials, or be used for passive treatments. Define the extent of all distinct lithologic types, including coal seams and their carbonaceous remnants, calcareous deposits, and potential water bearing zones.

The development of an ABR management plan should evaluate the range of potential solutions. The design process should be guided by sensible goals, and should include the mitigation plan should achieve the following goals:

- Low potential for long-term degradation to any watershed or public water supply.
- Reasonable level of confidence that acidic discharges will not develop in the future.
- Low level of long-term maintenance for treatment of any acidic discharge.

- Minimize delays and impact to the project construction schedule
- Minimize costs required for construction and maintenance associated with the ABR management plan, and the costs will be reasonable and prudent for addressing environmental concerns.

Developing a confident mitigation plan must always adequately consider not only the site specific geologic and hydrogeologic conditions, but also constructability. The list of factors that must be considered in developing an effective mitigation plan can be quite broad. These factors may include (but is not limited to) the following considerations:

- Sites with karst geology presenting a higher risk than non-karst limestone or non-carbonate sites.
- Sites with hydrologic connections to springs and/or streams
- Site proximity of existing private drinking water supplies, as well as proximity/availability of public drinking water sources and supply
- Sites designated “Unsuitable for Mining” under Pennsylvania’s Surface Mining Act may be off limits to disturbance due to existing regulations
- Potential for reclamation of nearby active or abandoned mine lands, elimination of safety or environmental hazards results in a beneficial use
- Proximity of project to sources of neutralization such as lime or waste-lime stockpiles
- The PA Chapter 93 water quality designation of the host watershed. Special Protection Waters, Exceptional Value (EV) or High Quality (HQ), are more sensitive than waters with less restrictive protected uses such as Cold Water Fishery or Warm Water Fishery. EV and HQ waters require a strict Alternatives Analysis to maintain the existing water quality under 25 Pa. Code § 93.4c, Implementation of Anti-Degradation Requirements. Non-discharge alternatives that are environmentally sound and cost-effective when compared with the cost of the proposed discharge must be considered first. If a non-discharge alternative is not available, the increased discharge shall use the best available combination of cost-effective treatment. For HQ waters, a reduction in water quality may be allowed if necessary to accommodate important economic or social development in the area in which the water is located.
- Conditions of the necessary transportation network for hauling ABR material (weight restrictions, detour requirements, etc.)
- Possible public acceptance or opposition of the ABR management plan

To fully cover all potential mitigation factors, it is necessary to consider both the material being excavated and any open (temporary and permanent) excavations or cuts (e.g. cut slope face). In effect, both cuts and fills may require some type of special considerations. While there are some similarities in treatment, the general approach for mitigation of excavated materials and exposed surfaces is sufficiently different that they will be discussed separately.

10.10.3 Treatment of Excavated Materials

Handling excavated material that is identified as potentially acid-bearing is primarily a material management issue. Identified ABR or sulfate soils should be “treated” as they are excavated. The goal is to prevent the acid reaction from starting and getting established. Remember that the reaction becomes self-feeding once the pH level has been sufficiently lowered. The more acidic the environment gets, the more rapid the reaction becomes. It should be recognized that flat-bedded or gently-dipping geologic deposits are usually more manageable during excavation than steeply-dipping deposits or veined deposits that typically are distributed in a rock mass that does not generally follow normal excavation sequencing. Since production rock blasting and subsequent excavation is done in successive downward blocks, horizontally oriented beds of ABR material can usually be readily separated from overlying and underlying deposits (as in surface coal mining).

It is important that the material of concern be adequately described and that appropriate personnel be on the project that can identify the material, so that the required handling and treatment procedures can be applied as appropriate and needed. The description must include the anticipated depth(s) and physical characteristics of the suspect material(s), and any other pertinent information. Identification during excavation is a critical factor in managing the material, to ensure that it is not used in uncontrolled fills, which can create problems later, and to effectively manage mitigation costs.

10.10.3.1 Excavated Rock

Treatment of excavated ABR involves all three of the primary tools discussed in [Section 10.10.1](#). See [Figure 10.10.3.1-1](#) for a typical detail indicating minimum requirements for a full-embankment section containing ABR fill. First, SAM is added as indicated in [Section 10.10.1.1](#). The SAM is added in the amounts determined in accordance with acid-base accounting procedures present in [Section 10.8](#), and must include a factor-of-safety of 2.0 for alkaline addition rates. The SAM is provided in the appropriate gradation and thoroughly mixed following the guidelines discussed in [Section 10.10.1.1](#) and [Table 10.10.1.1-1](#).

The treated ABR is encapsulated following the requirements of [Section 10.10.1.2](#). The treated ABR must be placed and encapsulated in a continuous process. To prevent infiltration of excess moisture from precipitation, any breaks in placement that will extend a duration exceeding five (5) days require that a Class-4, Type-A geotextile be placed followed by a minimum of two 6-inch compacted lifts of capping soil as defined in [Section 10.10.1.2](#). When the ABR fill is very coarse, angular and/or open-graded, the ABR must be choked off with granular material to prevent puncture of the geotextile.

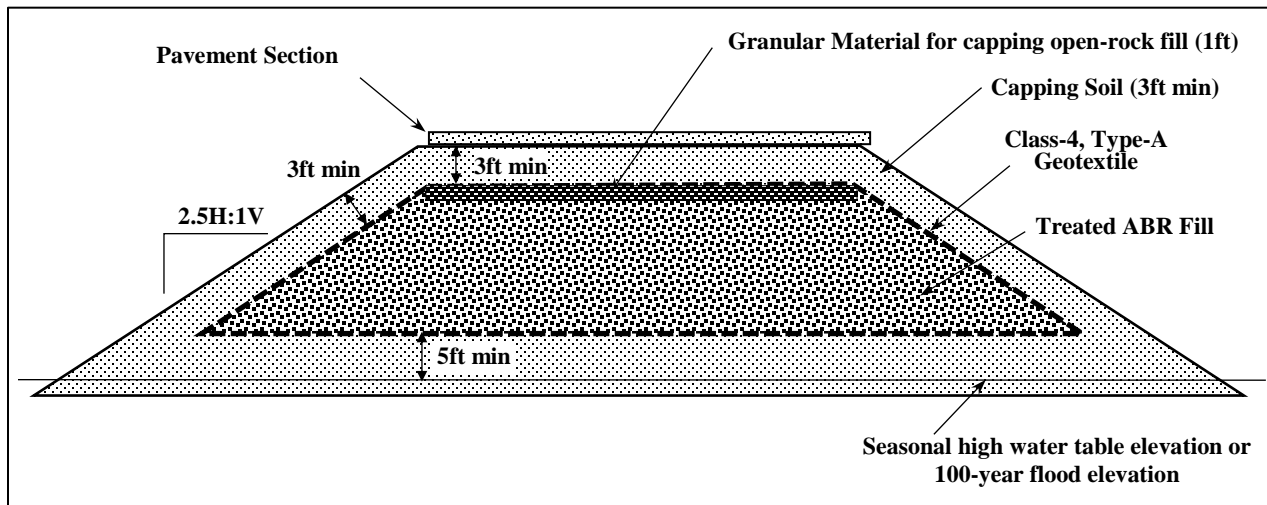


Figure 10.10.3.1-1 – Typical Section: Minimum Requirements for Treated ABR Fill

10.10.3.2 Excavated Soil

Excavated sulfate soils are treated similar to ABR by alkaline addition and encapsulation, except sulfate acid-bearing soils present two specific problems not experienced with ABR. The first is the rate of acid production. Because of the small particle size, acid production can initiate rapidly and progress very rapidly. Because of this, the alkaline addition serves much more as neutralization than simple buffering. The second concern is that the neutralization reaction can produce highly expansive minerals (e.g., gypsum). The specific gravity of gypsum is approximately 2.3 as opposed to a specific gravity of 4.9 to 5.2 for pyrite, the dominant sulfate soil mineral. The significantly lower specific gravity of the neutralization by-product results in an increase in volume from soil expansion, which can result in significant damage.

For this reason, it is impractical to use sulfate soils as engineered load-bearing fills or embankments requiring multiple lifts, since every lift of treated sulfate soils would have to be permitted to season, allowing the neutralization reaction and expansion to complete prior to compaction of the treated material. If the expansion process is not allowed to fully develop, or if portions of the sulfate soil are left untreated due to either insufficient alkaline addition or insufficient mixing of the treated soil, post construction damage can result. Therefore, sulfate soils must be treated (alkaline addition and encapsulation) and disposed of in waste areas.

Existing deposits of sulfate soils within the footprint of planned roadway construction, which cannot be avoided by realignment, must be removed, treated and wasted. Treatment is by alkaline addition and encapsulation. Alkaline addition rates are determined by acid-base accounting procedures, and must include a factor-of-safety of 2.0 for alkaline addition rates.

Sulfate soils can also be treated by submergence; however, this is considered a high-risk approach since it is likely the acid production will likely already have initiated prior permanent

placement below water. Treatment would still be necessary prior to placement, and then leaching of heavy metals into the groundwater may be a potential problem.

10.10.4 Treatment of Rock Excavated Faces (Rock Cuts)

Exposed excavations and rock cut slopes with ABR present a different challenge than handling the excavated materials. As with excavated material it is best to address the issue as soon as possible and practical after opening an excavation, as acid generation can begin to establish in as little as two to three weeks of the initial disturbance and exposure. Both temporary and permanent excavations should be addressed. The same principals of deny oxygen, deny moisture and alkaline addition apply to suspected ABR present in rock cuts. However, one significant difference between the open excavation and the material excavated is exposed surface area. Generally, the total available exposed surface area of the open excavation is much less than that of broken dislodged particles. This tends to reduce the potential oxidation rate. The obvious disadvantage is the greater difficulty that can exist in encapsulating a sloping, open excavation in order to further deny oxygen and moisture.

There are several approaches for treatment and mitigation of ABR and sulfate soil excavated faces. The selected treatment is primarily a function of the material (soil or rock) and the slope of the excavated face. The treatments may have some similarities, but also have specific differences, and therefore will be discussed individually in the following subsections.

Design and construct the rock cut slope in accordance with one of the methods (A through D) indicated in [Table 10.10.4-1](#) and discussed in detail below. Which method is selected will be determined according to a variety of site and project specific conditions and concerns. At a minimum, the following factors must be considered in assessing the appropriate cut slope design and construction procedures in ABR. These include:

- Is a rock cut slope in ABR avoidable? Have all possible alternatives been explored?
- Is the project earthwork balanced, borrow, or waste?
- Right-of-way requirements.
- Expected acid-bearing potential of excavated materials and rock cut slope faces.
- Anticipated rate and duration of acid production from exposed rock cut surfaces.
- Project scheduling – excavated ABR should be treated and encapsulated as soon as practical, so cuts and fills must be coordinated. Temporary storage of excavated materials must be avoided.
- Are any temporary or permanent measures required for control and/or mitigation of acid rock drainage?
- Are PA-DEP issued permits required such as a NPDES permit?
- Economic considerations (cost).
- The geologic structure (jointing and bedding) and rock slope stability of cut slope in ABR.
- Anticipated rock quality (weathering and degradation potential of rock cut faces in ABR).
- Anticipated rate and duration of acid production from exposed rock cut surfaces.

The last three factors are important in determining which of the methods is suitable relative to long-term control of acid rock drainage. The last factor, the duration of acid production from exposed rock cut faces, is important relative to the feasibility of being able to leave rock cut faces permanently exposed.

The method selected is often driven by cost, but also may be dictated by available right-of-way, environmental or other concerns. In general, Method A is appropriate for rock cut slopes in geologically favorable rock resistant to weathering, where acidic runoff can be managed for both short and long term conditions. Method B is for less competent rock, which may be subject to long term weathering and degradation, but limitations in right-of-way preclude flattening the slope sufficiently to permit a soil cover. And Method C is for cuts in much less competent or highly erodible rock, where adequate right-of-way exists to permit encapsulation of the cut with a soil cover. Methods B and C may also be applied if necessary for environmental needs or concerns, or if it is desired to limit the required amount of foreign borrow (however it is much more likely that the cost of foreign borrow is significantly less than treatment of ABR excavation).

Table 10.10.4-1 – Treatment of Rock Cut Excavations (Rock Cut Slopes)

Site Parameter/ Recommendation	Method A (Section 10.10.4.1)	Method B (Section 10.10.4.2)	Method C-1 and C-2 (Section 10.10.4.3)	Method D (Section 10.10.4.4)
Geologic Condition	No adverse discontinuities; high resistance to weathering; excellent to good rock quality	Highly fractured; moderate to low resistance to weathering; poor to very-poor rock quality	Moderately fractured; moderate resistance to weathering; fair rock quality	Isolated ABR seam within slope
Cut Slope Angle	1H:1V or steeper	1H:1V to 2H:1V with potential benches	2H:1V or flatter	1.25H:1V or flatter
Excavation	Presplit blasting techniques should be used on all ABR cut slopes			
Treatment of ABR waste rock	Treat excavated ABR in accordance with Section 10.10.3.1			
Treatment of ABR slope face	<p>1) Direct Weathering</p> <ul style="list-style-type: none"> - With no slope face coverage, the exposed ABR face is permitted to oxidize and “burn” itself out by forming an oxidized surface layer. <p>2) Shotcrete</p> <ul style="list-style-type: none"> - If the cut slope is completed in ABR with durable (sandstone) and low-durability units (shale, claystone, underclay, coal) consider applying shotcrete facing to the exposed low-durability units due to their high slaking potential. The shotcrete application has two advantages: a) it inhibits undercutting and b) it prohibits oxidation of the exposed ABR surface. - Provisions for stainless steel welded wire mesh, and associated hardware should be made. 	<p>1) Geosynthetic Facing</p> <ul style="list-style-type: none"> - Provide geosynthetic cover system on the ABR slope (see Figure 10.10.3.1-1). System to include combination of geotextiles, geomembrane and geocell. - To assure long-term stability, provide cabling through geocell; anchor at top of slope to support facing system. Cables, anchors and anchoring compounds must consist of highly acid-resistant, non-corroding materials. - Fill geocell with coarse calcareous aggregate to ballast system. 	<p>1) Capping Soil</p> <ul style="list-style-type: none"> - Provide four feet (min.) of soil covering for slopes in accordance with Figures 10.10.4.3-1 and 10.10.4.3-2. - Cut-slope benching required if slope steeper than 3H:1V. 	<p>1) Capping Soil</p> <ul style="list-style-type: none"> - Over-excavate outcrop of ABR seam. - Apply alkaline slurry treatment for additional prevention, if warranted - Provide four feet of soil covering for slopes in accordance with Figure 10.10.4.4-1. - Rock veneer required if slope steeper than 2H:1V.
Treatment of ABR runoff	Treatment types consist of short-term active and long-term passive systems and are designed based on water chemistry and amount of runoff generated; treatment systems for low level acid generation consist of open limestone drains, wetland or settling pond, and anoxic limestone drains. See Section 10.9 for additional information. In all cases a secondary (backup) passive treatment system should be considered to handle any potential fugitive acid production.			
Infrastructure Considerations	All piping should be thermoplastic pipe (HDPE); Do not use calcareous aggregates for drainage applications;			

10.10.4.1 Method A – Excavated Rock Face with Slope 1H:1V or Steeper

Because of the extreme difficulty and cost of attempting to encapsulate the face of a steep rock cut slope, an alternate strategy of addressing the problem can be considered. In [Section 10.2.4](#), the presence of oxidized cap rock was discussed. This is acid-bearing rock that has been previously oxidized, stripping it of its acid potential. During the oxidation process, an armoring of iron oxide (essentially rust) develops on the exposed rock surface that inhibits production of acid runoff. Recall that in order for a potential source of acidity to be converted to acid, the presence of both water and oxygen are necessary. One of the issues discussed in [Section 10.8.2](#) for alkaline addition is the importance of small particle size creating a large surface area for availability of the alkaline whereas larger particles develop an armoring on the surface of iron oxide (essentially rust) that inhibits future availability of the alkaline.

The concept is to use these properties in developing a plan for managing the exposed face of an acid-bearing rock cut slope. Once exposed, the face on the ABR cut slope can readily produce acid according to its potential acidity, and the availability of water and oxygen. While none of these three parameters can be directly controlled on a cut face, understanding the process allows for design of an effective management strategy.

Based on this information, the desired strategy is to simply allow the rock cut face to produce acid runoff, collect the runoff, and treat (neutralize by alkaline addition) the acid drainage prior to release. A rock cut slope completed in good quality rock, exhibits a limited surface area (as opposed to fragmented excavated rock). The limited surface area restricts the available potential acidity. Oxygen is abundantly available to the cut slope face while moisture availability varies. Slopes with high seepage will have greater availability of moisture. Precipitation events will also produce moisture. With the three necessary ingredients available (source of potential acidity, oxygen and water), acid drainage will be produced.

As the intensity of a precipitation event increases, acid drainage will be diluted. Detention basins and the treatment system must be designed with sufficient capacity so that a combination of treatment and dilution will result in release of water meeting required pH levels for a full spectrum of potential run-off rates. Treatment is often accomplished by metering of a liquid caustic (sodium hydroxide, NaOH) solution into the acid runoff. Metering rates will be determined by the pH level and flow rate of the runoff. Sodium hydroxide is frequently used since it has 100% neutralization efficiency, however any acceptable neutralizing agent with a properly designed system, is satisfactory.

Initially acid production from a rock cut face may be fairly high and the acidic runoff must be collected and treated prior to release. As the exposed sulfur bearing minerals in the rock are converted to acid, the level of available potential acidity continually decreases. Potential acidity “locked” in the interior of the rock is not exposed to air, and becomes much less available. An oxidized zone forms at the face of the slope, reducing the level of available acidity, and in effect protecting rock within the slope from further oxidation. The slope face essentially “burns” itself out. The slope will likely continue to produce acidic runoff, but at a rate and concentration that can likely be managed long term with a properly designed passive treatment system. Passive treatment systems are described in more detail in [Section 10.9.2](#).

A passive treatment system typically consists of a layer of limestone through which low level acidic runoff can flow and be neutralized prior to release of the project (refer to [Section 10.9.2.4](#)). This is most effectively and prudently placed in swales at the base of the cut slope. By placing in a swale directly beneath the rock formation producing the acidic runoff, the acid cannot permeate into surrounding soils, resulting in other potential problems (e.g. contamination of surrounding soils, degradation to pavements and pavement bases, damage to structure foundations, damage to light mast bases and/or foundations, damage to drainage structure, etc.)

Limestone is most effective in the smallest size practical; however, the particle size must be of sufficient size to avoid washout and erosion during intense precipitation events. A good approach to satisfy both these needs is to place a layer of finer graded limestone coarse aggregate (e.g., 2A or 2RC) to provide greater alkaline availability, with an adequate cover layer of coarser limestone aggregate (e.g., AASHTO No. 1's) or possibly limestone R-3 rock lining (if required) to dissipate flow energies and prevent erosion. The slower velocities created by the larger material in dissipating flow energies will also provide greater reaction time for neutralization.

The only drawback to a passive treatment system is the potential required maintenance. As remnant acid runoff from the slope is neutralized by the passive system, armoring of the limestone forms (essentially a “rust” coating as was discussed with alkaline addition), making the remaining alkaline unavailable.

Another important consideration is the selection of pipe material for and drainage features that may transport (internal exposure) or be subject to external exposure (permeating the surrounding fill) of acidic runoff or drainage. This issue is covered in greater detail in [Section 10.11](#) of this document. In short, neither metal nor concrete pipe fairs well in acidic conditions, and the use of approved plastic pipe (e.g., HDPE) is necessary for any areas where the potential for acid exposure exists.

10.10.4.2 Method B - Excavated Rock Face with Slopes Flatter Than 1H:1V and Steeper than 2H:1V

For rock slopes having a face flatter than 1H:1V, the opportunity exists to prevent (or drastically limit) the oxidation process and formation of acid by encapsulating the slope face. For slopes flatter than 1H:1V but steeper than 2H:1V, this can be accomplished with the use of a series of geosynthetic materials. The encapsulation denies both oxygen and water. Water may still reach the slope face from seepage, however exposure from external sources (precipitation and runoff from above) are controlled. And since the encapsulation also restricts availability of oxygen, any water entering as seepage will not have sufficient oxygen to produce acid in significant concentrations.

The encapsulation can be accomplished by placing a barrier or sequence of materials forming a barrier, which can seal off the slope face, but are durable and will provide long life. See [Figure 10.10.4.2-1](#) for a general cross-section and reference RC-78M, Slope Protection Geocell Cell and Geocell Section Details. The option presented here is to first dress the slope

face to as even and smooth a surface as practical. A smooth, even face will prevent air pockets and help protect the barrier system from damage and puncture.

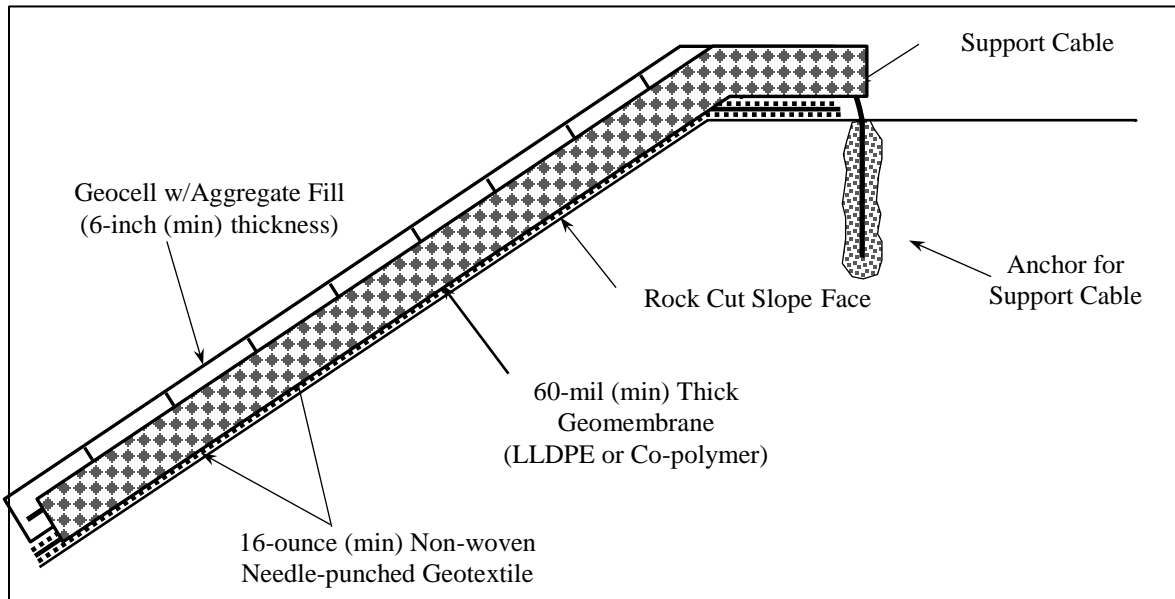


Figure 10.10.4.2-1 – Conceptual Detail for Isolation of Rock Cut with Slope Face Flatter than 1H:1V and Steeper than 2H:1V – (Method B)

The barrier should consist of a heavy weight non-woven needle-punched felt type geotextile, similar to a Class-4 Type-A geotextile, but of greater thickness. Class-4 Type-A geotextile weighs approximately 10 ounces per square yard of material (10 oz./sq. yd.). The barrier geotextile should have a minimum density of 16 ounces per square yard to provide adequate protection against puncture for the geomembrane layer above.

A geomembrane is then placed over top the geotextile layer. The geomembrane prevents infiltration of water and limits availability of oxygen. The geomembrane must consist of a flexible textured surface, minimum 60-mil thickness, linear low density polyethylene (LLDPE) or propylene/ethylene co-polymer membrane.

The geomembrane should be lapped top down (shingle style) and all seams field welded. A second geotextile (non-woven, needle punched, 16-ounce felt-type) is placed on top of the geomembrane to protect the geomembrane from puncture.

Perforated wall geocell is then placed over top the upper geotextile layer to protect and ballast the geotextile/geomembrane layers. The geocell will likely require a system of cables anchored at the top of the slope to support the geocell slope protection. The geocell is filled with an appropriate size coarse aggregate to provide ballast. Limestone aggregate should be used so that in the event of a puncture in the membrane, and the subsequent formation and leakage of acid occurs, a source of alkaline is available for immediate neutralization of the acid.

The dimensions (thickness and cell size) of the geocell, and size of the aggregate ballast, should be determined according to project specific needs, however the minimum thickness of the aggregate cover must be six inches.

Note that the procedure indicated in Method A can also be used for flatter slopes. Cost and environmental considerations may dictate which option is the preferred solution. Also, be aware that the slope may be producing acid runoff until the geosynthetic cap can be installed. Until that time, any acidic runoff must be managed as described in [Section 10.10.4.1](#) (Method A). Selection of pipes resistant to corrosive attack from acid (i.e., plastic pipe), for some (or all) of the project may also be necessary.

10.10.4.3 Method C - Excavated Rock Face with Slope 2H:1V or Flatter

For rock slopes with a cut face 2H:1V or flatter, an effective and relatively cost efficient soil encapsulation approach is possible (see Figures [10.10.4.3-1](#) and [10.10.4.3-2](#)). Slopes with a cut face steeper than 3H:1V should be benched to provide a good shear bond with the soil cover (Method C-1). Slopes shallower than 3H:1V should not require bonding benches unless specific site conditions or soil quality would dictate otherwise (Method C-2).

As with Method B, the soil encapsulation denies oxygen and restricts or limits water. Water may still reach the slope face from seepage or infiltration; however, with adequate soil cover thickness and proper vegetation, infiltration would be limited. And since the soil encapsulation restricts the availability of oxygen, any water entering (as seepage or infiltration) will not have sufficient oxygen to produce acid in concentrations that would be of concern.

A minimum of four feet of soil cover should be used to assure that vegetation root mass is not impacted by underlying acidic materials or byproducts. If the existing material is actively producing acid, or is of a highly acid nature, then a layer of pulverized agricultural lime must first be applied to the slope surface at a minimum rate of 10 pounds per square-yard to buffer and neutralize the existing material. The 10 pounds per square-yard of agricultural lime is a minimum required application rate. This rate must be increased as appropriate if site specific acid potential or anticipated/observed acid production rates dictate.

Once the capping soil is in place, it should be promptly seeded with acid-tolerant grasses such as standard Seed Formula-C (acid-tolerant to pH 4) or Formula-W (acid-tolerant to pH 5) in accordance with Publication 408, Section 804.2. To provide temporary erosion control until vegetation is established, the capping soil shall be properly mulched in conjunction with seeding in accordance with Publication 408, Section 805. If determined to be necessary or appropriate, soil amendments may also be included to aid in the growth and health of the site vegetation, and ultimately the site water quality. Horticultural, land management and related specialists can also be consulted for specifying other suitable vegetation options for specific site and soil conditions.

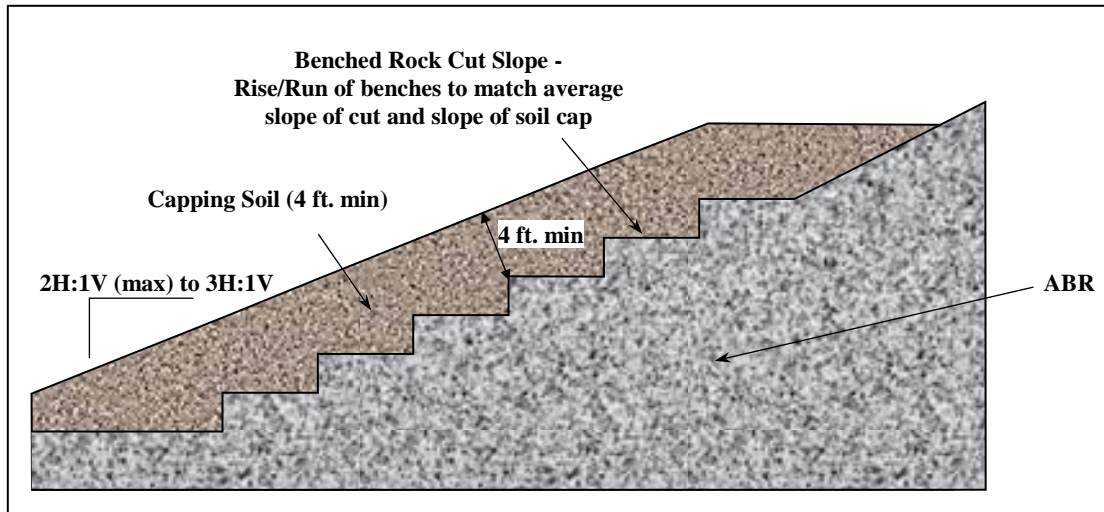


Figure 10.10.4.3-1 – Typical Section: Minimum Requirements Rock Cut with Slope Face Steeper than 3H:1V and Not Exceeding 2H:1V (Method C-1)

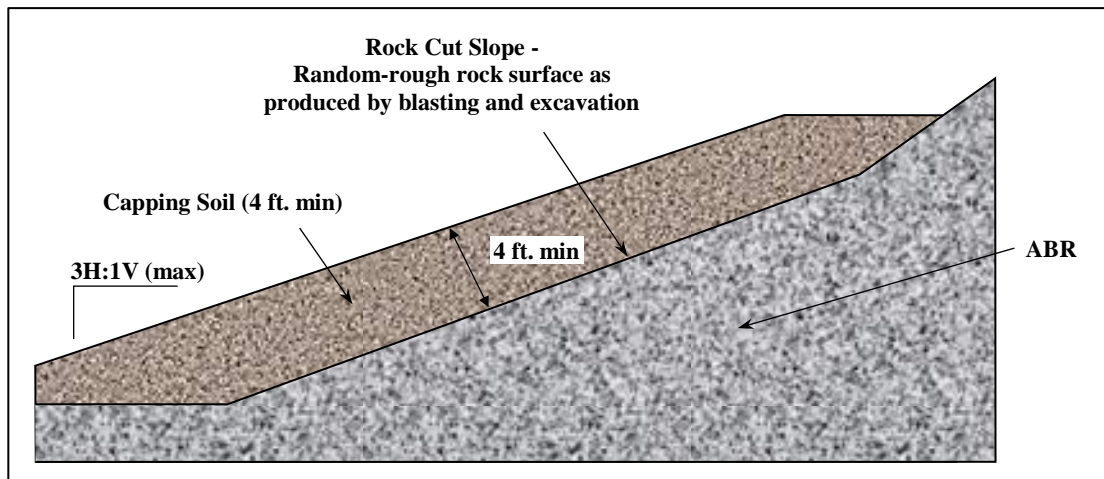


Figure 10.10.4.3-2 – Typical Section: Minimum Requirements Rock Cut with Slope Face 3H:1V or flatter (Method C-2)

Note that as with Method B, the slope may produce some short-term acidic runoff until the soil encapsulation can be completed. Any acid runoff and drainage must be managed as described in [Section 10.10.4.1](#) (Method A). Also, the treatments discussed in both Sections [10.10.4.1](#) (Method A) and [10.10.4.2](#) (Method B) can be applied for rock cuts with slopes 2H:1V and flatter, should specific site needs, environmental conditions or economic circumstances dictate.

10.10.4.4 Method D - Excavated Rock Face with ABR Seam Treatment - Slope 1.25H:1V or Flatter

For cut slopes 1.25H:1V or flatter that contain an isolated seam of ABR (e.g., black shale or coal) an effective method of preventing or minimizing acidic conditions is to over-excavate and cap the outcrop of the exposed seam, as shown in [Figure 10.10.4.4-1](#). For slopes 2H:1V and flatter, seam capping should specify soil backfill, with corresponding seeding and mulching. For cut slopes steeper than 2H:1V, seam capping should specify a veneer of Class R-4 rock over the capping soil (18-inch thickness, minimum) to provide long-term stability. Treatment Method-D is not recommended for slopes steeper than 1.25H:1V due to constructability and localized stability concerns.

Agricultural lime can be applied to the over-excavated surface prior to backfilling, similar to that specified in [Section 10.10.4.3](#) (Method C).

If groundwater seepage from the ABR seam is evident or anticipated, install an interception drain as shown in [Figure 10.10.4.4-1](#).

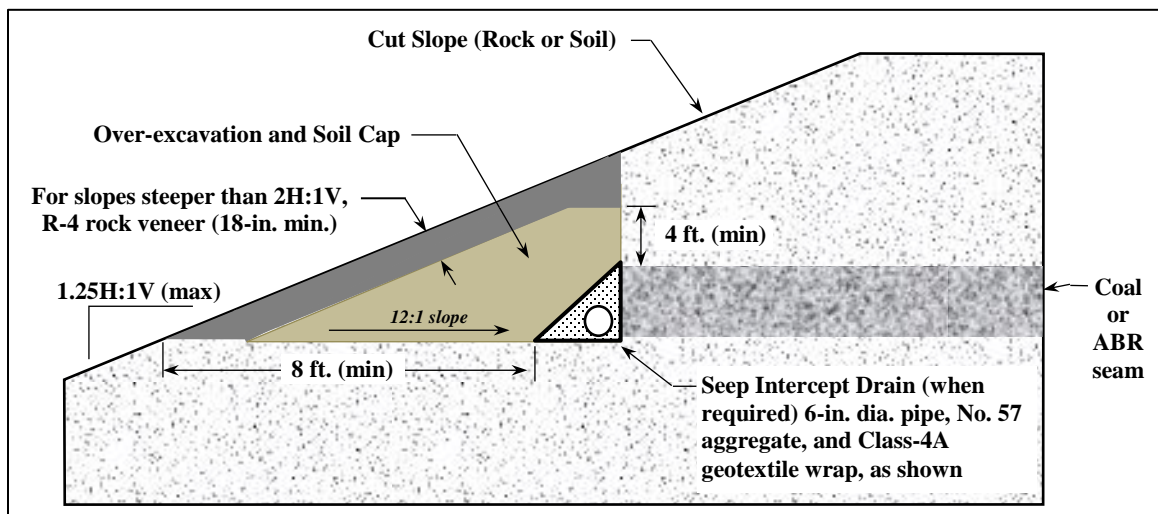


Figure 10.10.4.4-1 – Typical Section: Minimum Requirements Rock Cut with Seam Outcrop Treatment 1.25H:1V or flatter (Method D)

Note that as with Method B, the exposed ABR may produce some short-term acidic runoff until the soil capping can be completed. Any acidic runoff or drainage must be managed as described in [Section 10.10.4.1](#) (Method A).

10.10.5 Treatment of Existing Slope Soils and Seeps

Evidence of soils being excessively acidic may include oxide staining or lack of vegetation. Confirmation and quantification of soil acidity levels is determined by appropriate testing of representative soil samples. For existing embankment slopes and other soil deposits that are determined to be excessively acidic, surface treatments similar to that detailed in [Section](#)

[10.10.4.3](#) (Method C) are recommended. The same method options exist as described in [Section 10.10.4.3](#) (adding lime, soil, compost, seeding, and mulch), and can be used and adapted as site specific needs, environmental conditions or economic circumstances dictate and/or require.

For situations where sustained acidic seepage exists or is anticipated from a slope, covering the slope face with a geosynthetic drainage composite and geomembrane may be necessary. The drainage composite would capture the acid seepage, which would then be collected in an appropriately designed drainage system. The drainage system would convey the captured acidic runoff for treatment and eventual release. The geomembrane would provide a barrier limiting oxygen and infiltration of water, which could result in increased production and volume of acidic runoff.

10.10.6 Expansive Deposits – Pyritic Shales and Sulfate Soils

There are two mechanisms of expansion that are generally responsible for excessive volume changes in natural rock and soil materials:

1. Moisture adsorption (typically due to montmorillonite minerals)
2. Chemical alteration (typically due to sulfide or sulfate minerals)

The second mechanism is the subject of this section. Figure [10.10.6-1](#) illustrates the process.

Although materials that are potentially expansive due to moisture uptake can be a concern in Pennsylvania, they are not typically associated with ABR and therefore are not specifically addressed in this policy. Acid forming ABR materials usually contain sulfide minerals and therefore can be potentially expansive under certain conditions. One possible case is the addition of neutralizing lime to shale or soil containing sufficient levels of certain sulfate and clay minerals. This combination of materials could exhibit accelerated three-dimensional (3-D) swelling. Sulfate-induced ground heave can also occur when sulfide minerals oxidize to produce sulfate ions in the presence of clay minerals, moisture, and lime. This combination can then form problematic minerals such as gypsum or ettringite.

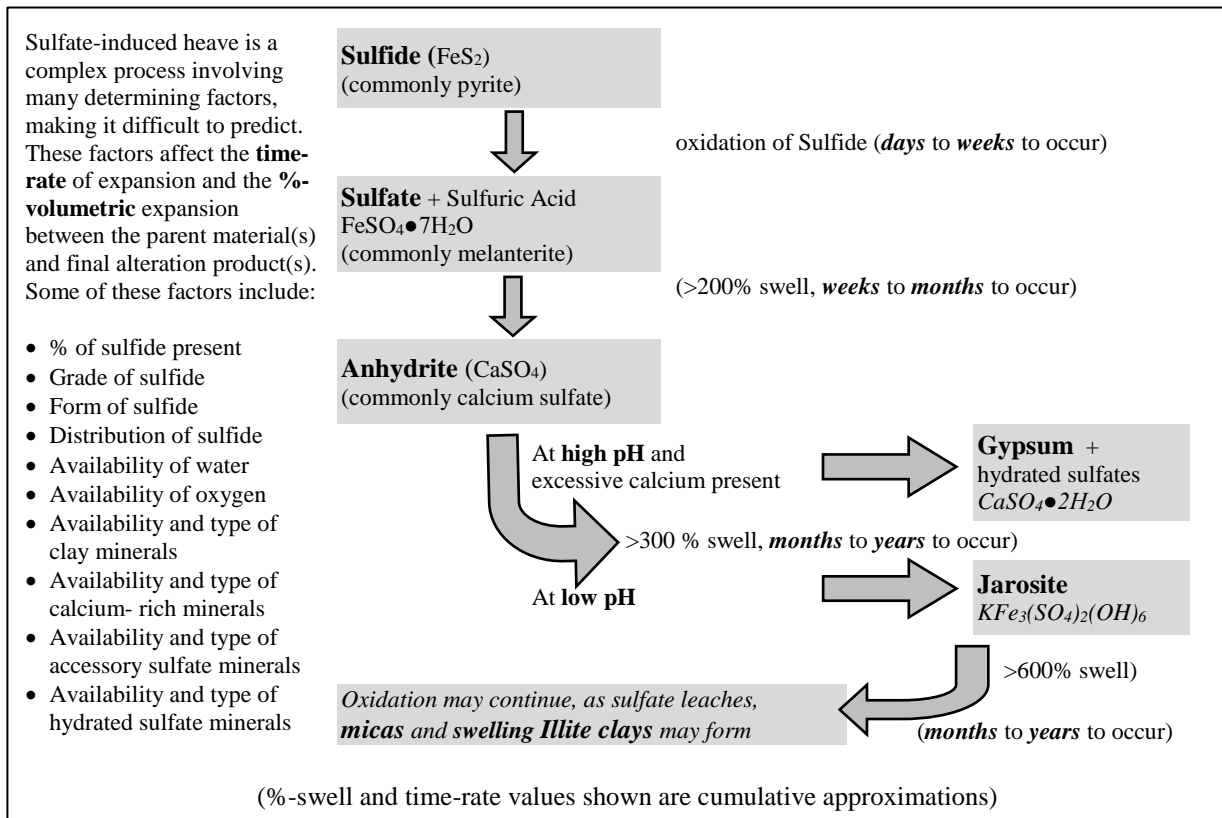


Figure 10.10.6-1 – Generalized Sulfide-Induced Swelling Process

When swelling shale is anticipated to be used as fill that is not encapsulated, a “mellowing” period shall be specified after the lime and water have been added and mixed with the fill to allow the sulfide-sulfate and sulfate-sulfate reactions to occur prior to final placement and compaction. Special attention must be given to using excess water during mixing, mellowing, and curing. Mixing water should be at least 3% above optimum for compaction, plus hydration water.

When common borrow is known or expected to possess elevated sulfate minerals, such as gypsum, the soluble sulfate level of the materials needs to be determined in accordance with test method AASHTO T-290. The action points for soluble sulfate, total sulfur, and sulfide sulfur levels are listed in [Table 10.10.6-1](#). It is known that for rock types other than coal, the %-total sulfur content is not expected to be significantly different than the %-pyritic (sulfide) sulfur content. For this reason, ASTM D-4239 is the specified test method for non-coal rock types. Conversely, sulfide sulfur may be a only a minor percentage of the total sulfur content in coal, and the forms of sulfur test method ASTM D-2492 is needed.

Table 10.10.6-1 – Action Points for Soil, Rock (other than Coal) or Coal Materials Treated with CaCO₃ or CO for the Control of Acid Generation

Sulfate Sulfur in Soil AASHTO T-290	Total Sulfur in Rock ASTM D-4239	Sulfide Sulfur in Coal ASTM D-2492	Potential for 3-D Volumetric Expansion	Required Action due to Potential Swelling
0 - 2,500 ppm (0 - 0.25%)	< 0.5%	< 0.5%	Very Low	None
2,500 - 5,000 ppm (0.25 - 0.50%)	0.5% – 1.0%	0.5% – 1.0%	Low	If material is to be used as roadway or structural fill, the material-alkaline blend must be tested in accordance with PTM No.130. During construction, a minimum 7-day mellowing period at +3% optimum moisture is required.
5,000 - 7,500 ppm (0.50 – 0.75%)	1.0% – 2.0%	1.0% – 2.0%	Moderate	
> 7,500 ppm (> 0.75%)	> 2.0%	> 2.0%	High	Do not use as structural or roadway fill due to excessive swell potential

Materials with a “low” or “moderate” swelling potential that are intended to be used as fill must be tested in accordance with PTM No.130. Materials of concern due to elevated **sulfate** levels (0.25% or greater) can be evaluated with the standard 14-day test duration. Materials of concern due to elevated **sulfide** levels (above 0.5%) however, shall be tested in accordance with PTM No.130 for a duration of 56-days. The extended test duration is required to permit the sulfide ample time to oxidize to sulfate prior to expansion. If PTM No.130 cannot be completed on these materials, or the results of PTM No.130 testing are uncertain or inconclusive, do not use such lime-treated fill in any area(s) where subsequent expansion or swelling would have a detrimental impact.

Caution must be taken to avoid the placement of lime-treated fill in a location that would be susceptible to intermittent saturation from seeps, springs or fluctuating groundwater that may contain elevated sulfate levels. These types of sulfate sources can trigger additional 3-D swelling in the alkaline-rich fill. In contrast, sulfide-bearing deposits that remain submerged (such as below a bridge footing adjacent to a water body) and undisturbed would not be expected to be potentially expansive or exhibit 3-D swelling because the oxygen supplied to the sulfide minerals is restricted.

10.11 DESIGN AND CONSTRUCTION CONSIDERATIONS

10.11.1 Design

Field Reconnaissance: The presence of efflorescent and/or reddish staining (termed “gossan”) on existing rock surfaces are revealing indicators of past acidic conditions where metal oxides have been formed. It should be well documented when these colorations are observed during the field reconnaissance or subsurface investigation.

Drainage Pipe Specification: The selection of pipe material type (ABS, aluminum, cement concrete, HDPE, steel, and PVC) is an important consideration. Refer to Publication 15M, Design Manual 4, Section 12.6.9 concerning requirements for pipes in corrosive environments. Do not specify metal or concrete pipe in areas where non-neutralized acidic fill or acidic drainage is expected or could occur. The acid will react with the alkaline in concrete pipe, and will rapidly oxidize the galvanized zinc coating of steel pipe. Use thermoplastic or vitrified clay pipe in any situation where highly acidic drainage is present or may occur, and PE-lined concrete or metal pipe for moderate to low acidic drainage as outlined in [Table 10.11.1-1](#) and Publication 15M, Section 12.6.9.

Table 10.11.1-1 - Selection of Pipe Material in Acid-Corrosive Environment¹

Pipe Material Type	Allowable Corrosivity Level of Soil and Water		
	pH	Resistivity (ohm-cm)	Sulfate (ppm)
Galvanized or Aluminized Steel	>5.5	>6,000	-
Concrete	>4.0	-	100-1,000
Aluminum	>4.0	>500	-
PE-lined Concrete, PE-lined Steel	-	-	-
Thermoplastic	-	-	-

Note: 1. Always reference the pipe manufacture specifications and Publication 15M, Section 12.6.9 for the various pipe material types to assure only pipes with adequate corrosion resistance (i.e., design life) will be permitted for the project-specific conditions.

Aggregate Specification: When alkaline addition is required, the use of limestone with a high CaCO₃ content (>90%) is preferred due to its notably higher reactivity than limestone with a high MgCO₃ or CaMg(CO₃)₂ content (i.e., dolomites). Calcareous aggregates may be used in designed treatment systems where aggregates have a determined design life, and will be removed and replaced with fresh aggregate at planned intervals. Class R-3 rock and AASHTO No. 1 aggregate sizes are considered optimal when long-term alkaline addition is desired for subsurface flow.

For subsurface drainage collection systems where the water conveyed is acidic or could become acidic and contains elevated ferric iron or aluminum, use only non-calcareous aggregates. Calcareous aggregates, such as crushed limestone, can react with these ions, causing metal precipitates to form and collect within the aggregate and the drainage pipes. This precipitate can quickly accumulate and completely clog the drain system.

For surface drainage collection systems, calcareous aggregate and rock can be used effectively to line ditches and slopes to provide alkalinity to buffer acidic surface water and rainwater at a project site.

10.11.2 Construction

When acid-producing rock is expected or suspected to be encountered during construction, plan to have qualified personnel on the project to perform timely identification and assessment of the material during excavation. This is especially important for projects within areas of suspected isolated sulfide deposits that are typically difficult to locate and define accurately during the drilling program. In such cases, a project-specific contingency plan should be developed by the Department and be in-place prior to the start of construction. The plan must address the prevention and/or mitigation of ARD during planned excavations should unexpected acid bearing rock be encountered. The plan should be reviewed with PA-DEP prior to finalizing project plans and specifications and bidding of the project.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

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CHAPTER 11 – GEOSYNTHETIC REINFORCED SOIL (GRS) SLOPES

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11.1 INTRODUCTION

This chapter provides design requirements and guidelines for GRS slopes on Department roadway and structure projects. Standard Drawing RC-14 provides typical details and notes for GRS slopes, and Department Standard Special Provisions must be used for construction of GRS slopes.

Geosynthetic reinforced soil (GRS) slopes consist of horizontal layers of geosynthetics placed within the soil used to construct the embankment. These reinforcements allow the embankment slope to be constructed steeper than typical embankment slopes (i.e., 1.5(H):1(V) or flatter). Embankments constructed with geosynthetics can have slopes as steep as 0.25(H):1(V). GRS slopes reduce the footprint of embankments, which can be beneficial for several reasons, including: limiting right-of-way, minimizing impacts to wetlands, and avoiding encroachment onto roadways or other facilities. GRS slopes are typically less costly than retaining walls and require less maintenance.

11.2 REINFORCED SLOPE DESIGN REQUIREMENTS

GRS slopes must be designed using the design requirements included in this chapter, and must be constructed according to the Department's Standard Special Provision cXXXXX ITEM 9200-0200 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE A; ITEM 9200-0201 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE B; ITEM 9200-0202 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE C; ITEM 9200-0203 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE D; ITEM 9200-0205 EXCAVATION OF UNSUITABLE FOUNDATION MATERIALS and Standard Drawing RC-14. GRS slopes greater than 40 feet in vertical height require review and approval by the Chief Geotechnical Engineer.

Stability of a GRS slope depends on the reinforced soils ability to act as a stable but flexible mass, withstanding all internal and external loads without failure, loss of ability to adequately support overlying pavements or structures, or otherwise perform its intended function. The GRS slope must be designed to provide the required levels of internal and external stability and for all of the potential failure modes. The specified minimum factors of safety are required for both the short-term and long-term conditions, and for all failure modes.

Slope stability analyses must be done for all failure modes using the latest version of SLIDE7, or with hand calculations when appropriate (e.g., horizontal sliding). Complete the analyses assuming that the orientation of the geosynthetic tensile force is in the plane of the geosynthetic (i.e., horizontal), and use an inclination factor of zero in SLIDE7. When specifying tensile strength of the primary geosynthetic, define the primary geosynthetic strength in SLIDE7 using a minimum of four points as shown in [Figure 11.2-1](#), except when wrapped faces or ends are used. This is necessary to meet the boundary conditions of $T = 0$ at both ends of the primary geosynthetic. Also, use minimum embedment lengths, L_e , of two feet at the slope face and at the end of the primary geosynthetic in order to model the developed tensile forces as shown [Figure 11.2-1](#).

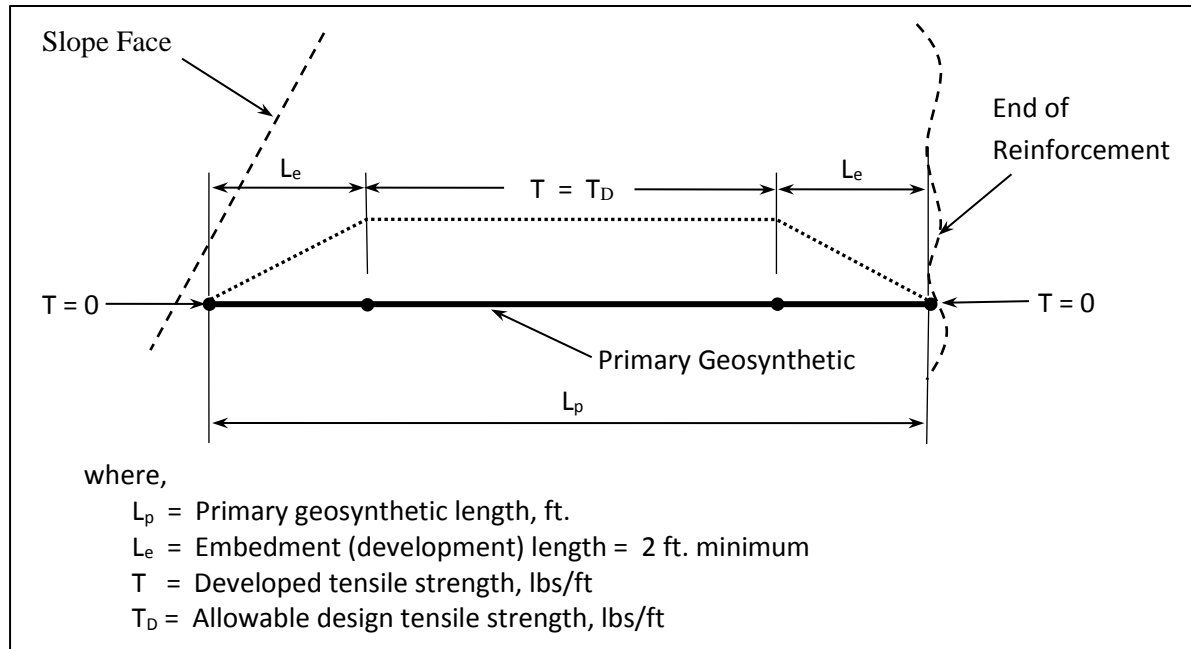


Figure 11.2-1 – Primary Geosynthetic Tensile Strength Distribution

11.3 GRS SLOPES IN FLOODPLAINS AND SUBSURFACE DRAINAGE

A GRS slope can be designed to be within a floodplain provided the proper groundwater conditions are modeled in the slope stability analyses, and the limitations relative to floodplain conditions cited elsewhere in this chapter are followed. Note that Type A GRS slopes are not permitted within a 500-year floodplain. To prevent excess moisture and saturation of the reinforced fill, adequate subsurface drainage shall be specified for slopes constructed within a floodplain, or other areas where water is anticipated behind the GRS slope is anticipated. This is especially critical for reinforced slopes constructed against existing slopes that have seepage or springs. Slopes constructed against or over seeps or springs must have an open-graded drainage gallery consisting of AASHTO No. 8 or AASHTO No. 57 Coarse Aggregate, collection pipes (as necessary), and Class 4, Type A geotextile separator between coarse aggregates and soils. The drainage gallery must have a positive outlet.

Below the 500-year flood elevation, specify granular fill for the reinforced mass consisting of AASHTO No. 8 Coarse Aggregate meeting the requirements of Publication 408, Section 703. When using No. 8 Coarse Aggregate reinforced fill, wrap the Class 4, Type A geotextile secondary geosynthetic around the No. 8 Coarse Aggregate, at both the slope face and at the back of the reinforced fill. Additionally, provide a Class 4, Type A geotextile on the prepared foundation to prevent migration of fines into the No. 8 Coarse Aggregate. The secondary geosynthetic wraps must be embedded a minimum of four feet into the next geosynthetic layer. For layers of reinforced fill not underlain by secondary geosynthetic (i.e., at layers of primary geosynthetic), use a wrap of secondary geosynthetic embedded a minimum of four feet, both top and bottom, at both the face and at the back of the slope. Depending on the slope type, if necessary, provide a front face drainage collection system to drain the water, or grade fill to drain to the back face of the slope, and provide a collection system that outlets the

drainage. The supplemental drainage system must allow free flow of water from the coarse aggregate fill, without resulting in loss of material.

If specifying GRS slopes within a 500-year floodplain, or in areas subject to inundation or swiftly flowing surface water (flow velocity greater than three feet per second), perform hydraulic analysis and provide appropriate sized rock lining in accordance with DM-4, Part A, 2015, Section 7.2.5.

11.4 GRS STABILITY AND FOUNDATION SUPPORT

Design requirements outlined in this chapter require that the slope will be constructed on a stable foundation. Total and differential settlement, and local bearing failure at the toe (i.e., lateral squeeze) of the proposed GRS slope must be evaluated. Additionally, the shear strength of the foundation soil must be adequate to support the proposed GRS slope. If a weak layer exists beneath the GRS slope to a limited depth which is less than the width of the slope, the factor of safety against failure by squeezing may be calculated by the following equation:

$$FS_{squeezing} = \frac{2c_u}{\gamma_{rf}D_s \tan\beta} + \frac{4.14c_u}{H\gamma_{rf}} \geq 1.3$$

where,

- β = angle of slope
- γ_{rf} = unit weight of reinforced soil fill
- D_s = depth of soft soil beneath slope base of the embankment
- H = height of slope
- c_u = undrained shear strength of soil beneath slope

Caution is advised and a more thorough analysis should be performed when the factor of safety against failure by squeezing is less than two. When the depth of the soft layer is greater than the slope base width, general slope stability will govern the design.

Extreme force effects are to be determined for each design requirement by using the maximum or minimum unit weight to calculate the controlling condition. The use of the maximum or minimum load (unit weight) depends on the applied load direction and the design criteria under consideration. For example, for a sliding calculation, the minimum unit weight should be used to provide the most conservative vertical force.

In general, the foundation materials must be capable of supporting the GRS slope and all appurtenant features associated with the GRS slope without failure or unacceptable settlement of the foundation materials. Treatment and/or removal and replacement of foundation materials are options to increase the bearing resistance and stability of the foundation. Potential foundation settlement must also be evaluated relative to its impact on:

- drainage system and slope facing of the reinforced fill
- facilities constructed within or on the reinforced fill
- adjacent structures and drainage.

As a flexible structure, the controlling concern related to settlement will likely be with any facilities (e.g., pavement structure, drainage, utilities, etc.) the reinforced fill is supporting. If the magnitude of predicted post-construction settlement exceeds 1 inch, provisions must be made in the overall design to accommodate these movements.

11.5 SLOPE FACE TYPE

Four slope face types, designated Type A through Type D, can be used. A discussion of these faces including limitations on their use are provided below, and typical sections are included in Standard Drawing RC-14.

11.5.1 Type A Slope – Vegetated

Type A slope facing consists of stepped wire mesh construction forms with a Turf Reinforcement Mat (TRM) backing and a fertile soil lining. The TRM backing functions to retain soil until vegetation is established. The Type A slope may be specified and constructed for slopes no steeper than 0.5(H):1(V). Type A Slopes are not permitted within a 500-year floodplain.

In order to establish and support healthy vegetative cover, a nominal width of 12 inches of fertile soil must be provided at the slope face, directly behind the turf reinforcement mat and wire form. The fertile soil must meet the requirements of Publication 408, Section 206.2(a)1.1.a, except that it must have greater than 25 percent passing the No. 200 sieve. The soil must be capable of supporting and maintaining plant growth. If the material used for the reinforced slope fill also meets these requirements, then this material can be used behind the TRM and wire mesh form. Specify a TRM meeting the requirements of Publication 408, Section 806.2(b).

The vegetated slope face provides long-term erosion control. Specify Formula-C (Crown Vetch and Annual Ryegrass) seeding mix per Publication 408, Section 804.2(b). Vegetated reinforced slopes must be protected against application of herbicide treatments. Specify mulch per Publication 408, Section 805.2(a)1.e, Fiber Bonded Matrix.

11.5.2 Type B Slope – Rock Lined with Wire Construction Forms

Type B slope facing consists of stepped wire mesh construction forms with a Class 4, Type A geotextile backing, a layer of AASHTO No. 1 Coarse Aggregate (i.e., aggregate lining) on the slope face, and a durable rock lining. The rock lining shall be sized as appropriate for project conditions. Refer to [Section 11.3](#) for requirements when constructing GRS slopes in floodplains. The minimum size of rock lining is Class R-4 when the slope is not located within a floodplain. Minimum specified thickness of rock lining is 2.5 times the top size of the rock lining class specified, measured perpendicular to the slope face. Specify a 12 inch cushion layer of AASHTO No. 1 Coarse Aggregate to be placed over the stepped face prior to placement of the rock lining. The geotextile functions to retain finer aggregates and soil. The Type B slope may be specified and constructed no steeper than 1(H):1(V).

11.5.3 Type C Slope – Rock Lined with Geotextile Wrapped Face

Type C slope facing consists of a stepped geosynthetic wrapped face, a layer of AASHTO No. 1 Coarse Aggregate (i.e., aggregate lining) on the slope face, and a durable rock lining. The rock lining shall be sized as appropriate for project conditions. Refer to [Section 11.3](#) for requirements when constructing GRS slopes in floodplains. The minimum size of rock lining is Class R-4 when the slope is not located within a floodplain. Minimum specified thickness of rock lining is 2.5 times the top size of the rock lining class specified, measured perpendicular to the slope face. Specify a 12 inch cushion layer of AASHTO No. 1 Coarse Aggregate to be placed over the stepped face prior to placement of the rock lining. The Type C slope may be specified and constructed no steeper than 1.25(H):1(V).

11.5.4 Type D Slope – Gabion

Type D slope facing consists of stepped wire-mesh gabions. When used in a floodplain, the gabion facing must be protected by a layer of AASHTO No. 1 Coarse Aggregate (i.e., aggregate lining) and a layer of durable rock lining up to the 500-year flood elevation. The rock lining shall be sized as appropriate for project conditions, and shall not be placed at a slope steeper than 1(H):1(V). Provide a horizontal bench at the top of the rock lining of a minimum width equal to the required rock lining placement thickness. Refer to [Section 11.3](#) for requirements when constructing GRS slopes in floodplains. The minimum size of rock lining is Class R-4. Minimum specified thickness of rock lining is 2.5 times the top size of the rock lining class specified, measured perpendicular to the slope face. Specify a 12 inch cushion layer of AASHTO No. 1 Coarse Aggregate to be placed over the stepped face prior to placement of the rock lining. The Type D slope may be specified and constructed no flatter than 0.5(H):1.0(V) and no steeper than 0.25(H):1.0(V). Specify corrosion resistant, Type B (i.e., hand-placed rock only) gabions meeting the requirements of Publication 408, Section 626.

Specify a Class 4, Type A geotextile backing meeting Publication 408, Section 735 as a separator behind each course of gabions to guard against potential piping or loss of soil from the reinforced mass. The geotextile backing is wrapped as shown in Standard Drawing, RC-14 providing continuous coverage of the rear face of each course of gabions and adequate anchorage.

11.6 GEOMETRIC, LOAD, AND PERFORMANCE REQUIREMENTS FOR DESIGN

11.6.1 Establishing Geometric Requirements

The GRS slope height, H , is equivalent to the total height of the slope. The height used for design includes any embedment into the foundation soil, but the embedment must be ignored for stability and sliding calculations. The GRS slope angle, β , is the angle of the slope face in degrees measured from horizontal. The anticipated depth to groundwater, d_w , must be modeled based on information obtained from the subsurface exploration. Note that one of the functions of the secondary geosynthetic is to provide drainage within the reinforced fill. [Figure 11.6.1-1](#) also defines slope height, slope angle, and groundwater depth.

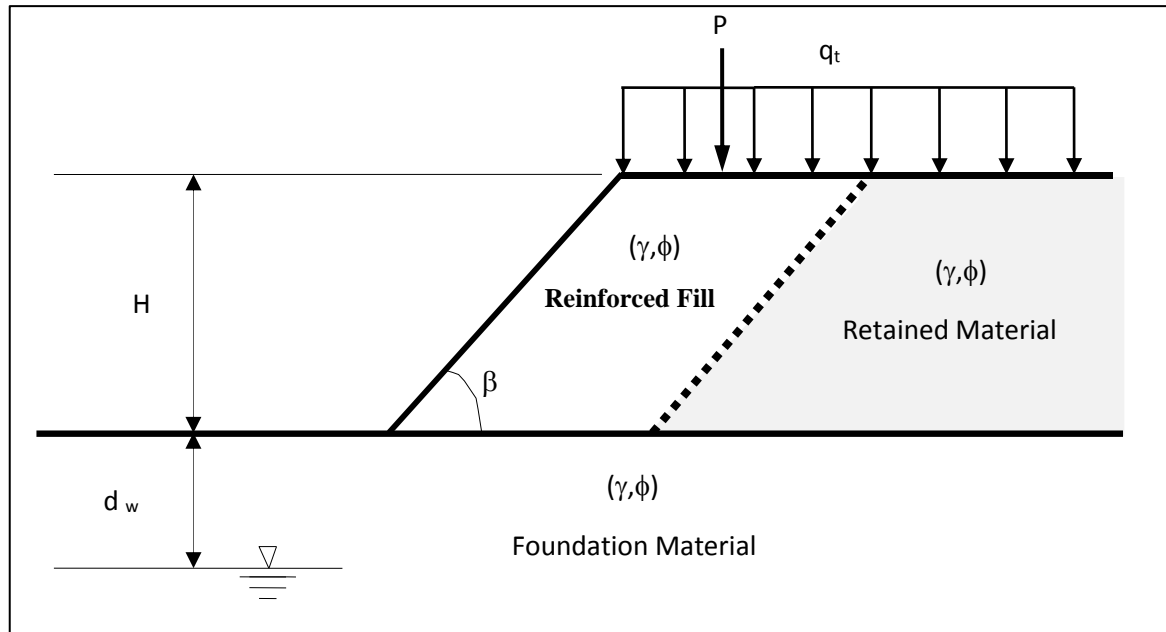


Figure 11.6.1-1 – Geometric and Loading Requirements for GRS Slope

11.6.2 Establishing Load Requirements

11.6.2.1 Surcharge Loads

For slopes with traffic live loads on top, apply a minimum traffic surcharge load, (q_t), of 360 pounds per square foot (psf). Apply the traffic surcharge to the top of the slope starting at the face edge of the reinforced slope, extending to at least the furthest distance that any future traffic loads can be reasonably expected for the life of the slope (i.e., 100 years) with the proposed design geometry. Minimum width of the surcharge load is the greater of:

- required length of the primary geosynthetic
- height of the slope
- furthest point from the slope face to any paved surface, including paved and unpaved shoulder areas
- distance between the top of slope and the furthest distance that any future traffic loads can be reasonably expected for the life of the slope.

Apply larger magnitude surcharge loads or other surface loads if project specific conditions dictate. In all other cases use a minimum surcharge load of 240 psf. Apply the minimum design surcharge to the top of the slope starting at the face of the slope, extending to at least the furthest distance that construction or construction earth surcharge loads can be reasonably expected with the proposed design geometry, but at a minimum to the limits of the geosynthetic reinforcement.

11.6.2.2 Point Loads

Point loads, P, such as from a sign or light structure, must also be included in the design. Apply the load at the anticipated location and with the appropriate magnitude.

11.6.3 Establishing Performance Requirements

Performance (i.e., factors of safety) of GRS slopes must be checked for four stability modes, including:

1. Horizontal Sliding Failure (FS_S) – sliding of the reinforced soil mass along its base
2. Deep-Seated Failure (FS_{EXT}) – failure surfaces passing behind and beneath the reinforced soil mass
3. Compound Failure (FS_{COM}) – failure surfaces passing through both the reinforced soil mass and foundation and/or retained soils
4. Internal Failure (FS_{INT}) – failure surfaces passing entirely within the reinforced soil mass

The minimum required factors of safety for the various stability modes are listed in [Table 11.6.3-1](#), and the different stability modes are shown in [Figure 11.6.3-1](#).

Table 11.6.3-1 – Minimum Required Stability Factors of Safety (FS_R)

Stability Mode		Required Factor of Safety ¹	
		Slopes 1:1 and flatter	Slopes steeper than 1:1
External	Horizontal Sliding (FS_S)	2.0 ¹	2.0 ¹
	Deep-Seated (FS_{EXT})	1.3	1.5
Compound (FS_{COM})		1.3	1.5
Internal (FS_{INT})		1.3	1.5

Notes: 1. If internal angle of friction (i.e., phi) values are obtained from laboratory shear strength testing (i.e., direct shear or triaxial shear) conducted on representative samples of the foundation soil, retained soil and the reinforced fill, a minimum factor of safety against sliding of 1.5 is permitted. When conducting laboratory tests on remolded samples of representative material, remold samples so that the stress applied during remolding does not exceed the confining stress to be applied during testing on each sample increment.

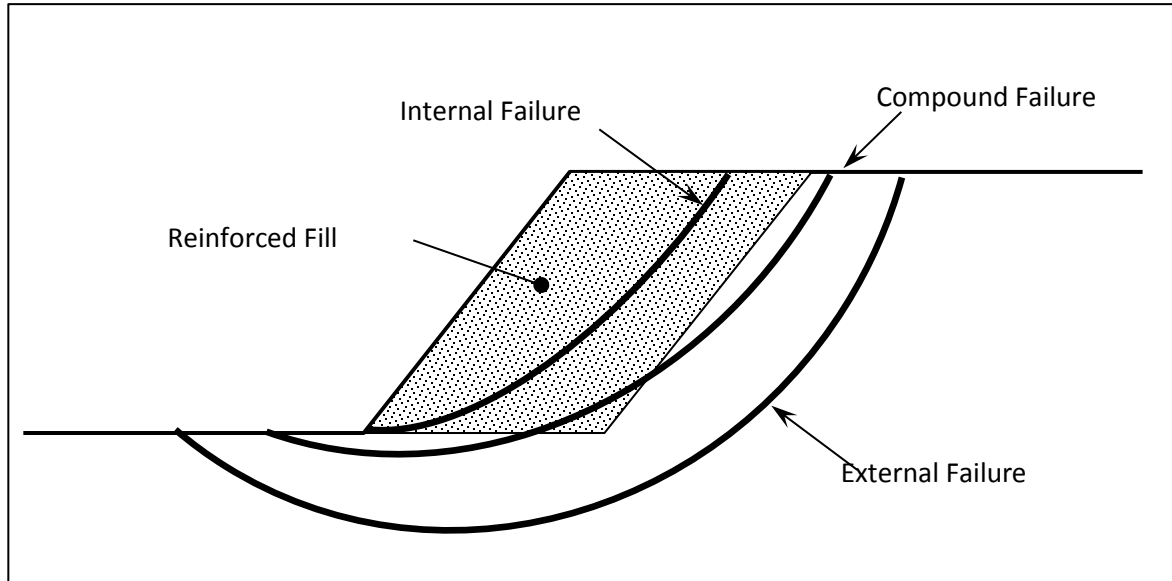


Figure 11.6.3-1 – Failure Modes

11.7 EARTH MATERIAL REQUIREMENTS

11.7.1 Foundation Materials

Determine and provide all foundation information, including subsurface profile, shear strength parameters, unit weight, and settlement calculations in the Geotechnical Engineering Report (GER). Determine the shear strength parameters from laboratory shear strength testing of these materials in order to utilize an external horizontal sliding safety factor of 1.5. Allow for the variability of the foundation materials in assigning shear strength parameters for design. Use peak shear strength parameters in analyses, unless conditions require the use of residual shear strength parameters (e.g., an existing failure in the foundation soil, in-situ weak layers/varves, etc.). Determine shear strength parameters using direct shear ([ASTM D 3080](#)) or consolidated-undrained (CU) triaxial tests with pore pressure measurement ([ASTM D 4767](#)). A nominal cohesion value of up to 200 psf may be used for foundation materials where appropriate and justified by laboratory testing demonstrating either a minimum Plasticity Index (PI) of seven or supported by laboratory shear strength testing.

If for any reason laboratory shear strength testing cannot be performed on the foundation materials, use an external horizontal sliding safety factor of 2.0.

11.7.2 Retained Materials

Determine and provide all retained materials information, including subsurface profile, shear strength parameters, unit weight and settlement calculations (if applicable), in the GER. If the retained materials are known (e.g., in-situ soils/embankment, proposed fill constructed from known/on-site soils, etc.), determine the shear strength parameters from laboratory shear strength testing of these materials in order to utilize an external horizontal sliding safety factor of 1.5. Allow for the variability of the retained materials in assigning shear strength parameters for

design. Use peak shear strength parameters in analyses, unless conditions require the use of residual shear strength parameters, such as an existing failure in the retained materials. Determine shear strength parameters using direct shear (ASTM D 3080) or consolidated-undrained (CU) triaxial tests with pore pressure measurement (ASTM D 4767). A nominal cohesion value of up to 200 psf may be used for retained materials where appropriate and justified by laboratory testing demonstrating either a minimum Plasticity Index (PI) of seven or supported by laboratory shear strength testing.

If for any reason laboratory shear strength testing cannot be performed on the retained materials, or if the retained materials are not known at the time of design, an external horizontal sliding safety factor of 2.0 must be used for design unless laboratory shear strength testing of the retained material is performed during construction. If shear strength testing is deemed necessary, the Department's Standard Special Provision cXXXXX ITEM 9200-0200 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE A; ITEM 9200-0201 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE B; ITEM 9200-0202 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE C, ITEM 9200-0203 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE D; ITEM 9200-0205 EXCAVATION OF UNSUITABLE FOUNDATION MATERIALS must be amended to include requirements for minimum shear strength parameters for the retained materials and shear strength testing requirements (i.e., similar to the requirements for the reinforced fill).

11.7.3 Reinforced Fill Materials

The reinforced fill material must meet requirements for internal angle of friction, gradation, pH, and unit weight.

11.7.3.1 Internal Friction Angle

Specify the required minimum internal friction angle for the reinforced fill in the Department's Standard Special Provision cXXXXX ITEM 9200-0200 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE A; ITEM 9200-0201 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE B; ITEM 9200-0202 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE C, ITEM 9200-0203 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE D; ITEM 9200-0205 EXCAVATION OF UNSUITABLE FOUNDATION MATERIALS. The required minimum internal friction angle of the reinforced fill must be assessed relative to design constraints and objectives. In assigning this required minimum value, select a value that is consistent with any materials that must be excavated to construct the reinforced slope, materials readily available on-site, or materials from local borrow sources if on-site materials are unavailable. For economic considerations, manufactured aggregates, select granular fill, or other special gradation or high shear strength materials should only be specified when indicated elsewhere in this chapter or required by site conditions and/or design objectives.

Establish the minimum reinforced fill internal friction angle required for construction. When the design is based on the use of existing available project soils or a known off-site borrow source (i.e., known materials), determine the internal friction angle from laboratory shear strength testing of these materials. Allow for the variability of the on-site materials in assigning

required minimum internal friction angle. Use peak internal friction angle in analyses. Determine peak internal friction angle of reinforced fill in accordance with AASHTO T 236 on the portion finer than the 2.36 mm (No. 8) sieve. Prepare the sample by substituting material coarser than the No. 8 sieve with material passing the No. 8 sieve but retained on the No. 10 sieve. The percentage of all fractions smaller than the No. 10 sieve shall remain proportioned as the original sample prior to substitution. Shear strength testing of reinforced fill material consisting of AASHTO No. 8 Coarse Aggregate is not required.

When the source of reinforced fill material is unknown or is not available for testing, specify a minimum friction angle that must be met for the reinforced fill. It is very important that the minimum value specified can be met by locally available natural earth materials meeting the gradation and pH requirements. In some cases this may result in a relatively low specified friction angle (i.e., as low as 24 degrees) if generally low quality natural materials are locally available. **Do not specify a value that is inconsistent with any materials that must be excavated to construct the reinforced slope, materials readily available on-site, or materials from local borrow sources if on-site materials are unavailable, unless a specific design need exists for such material that cannot be satisfied with primary geosynthetic strength or length adjustments.**

The maximum design internal friction angle (ϕ) value for reinforced fill cannot exceed 36 degrees, regardless of laboratory shear strength test results. The use of cohesion in the analyses is not allowed for the reinforced fill (i.e., $c = 0$).

Note that if an unknown borrow source will be used to obtain the reinforced fill, and consequently no laboratory shear strength testing is performed during design, an external horizontal sliding safety of 1.5 can still be used if laboratory shear strength testing is performed on the foundation and retained materials, since laboratory shear strength testing on the reinforced fill will be performed during construction. For portions of the GRS slope that use AASHTO No. 8 Coarse Aggregate for backfill, use a design friction angle of 36 degrees.

11.7.3.2 pH

In order to establish and support healthy vegetative cover for Type A slopes, a nominal width of 12 inches of fertile soil must be provided at the slope face, directly behind the turf reinforcement mat and wire mesh form. The fertile soil must meet the requirements of Publication 408, Section 206.2(a)1.1.a, except that it must have greater than 25 percent passing the No.200 sieve. The soil must be capable of supporting and maintaining plant growth. Unless otherwise required for special plantings, such as wetlands, provide soils within a pH range of 5.5 to 7.5 determined in accordance with AASHTO T 289.

For all other areas of Type A slopes, and the entire fill for all other slope types, specify backfill consisting of natural material, having a pH between 5.5 and 9.0 determined in accordance with AASHTO T 289.

11.7.3.3 Gradation

The gradation of the reinforced fill material must meet one of the following:

- Publication 408, Section 206.2(a)1.a, Soil, except particles greater than 4 inches must be removed prior to compaction.
- Well graded granular material meeting the following gradation requirements:

Sieve Size	Percent Passing
50 mm (2 inch)	100
4.75 mm (No. 4)	50 – 100
425 μm (No. 40)	15 – 55
75 μm (No. 200)	0 – 35

- Publication 408, Section 703.2, AASHTO No. 8 Coarse Aggregate, where required.

11.7.3.4 Unit Weight

The acceptable ranges of unit weights for soil and aggregate reinforced fill are included in the Department’s Standard Special Provision cXXXXX ITEM 9200-0200 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE A; ITEM 9200-0201 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE B; ITEM 9200-0202 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE C, ITEM 9200-0203 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE D; ITEM 9200-0205 EXCAVATION OF UNSUITABLE FOUNDATION MATERIALS. Since the “exact” unit weight of the reinforced fill will not be known until construction, design the GRS using a total unit weight for reinforced fill of 120 pcf. When GRS slopes are located in a floodplain, requiring AASHTO No. 8 Coarse Aggregate for reinforced fill, use a unit weight of 90 pcf for the coarse aggregate zone of the GRS slope.

11.8 GEOSYNTHETIC REQUIREMENTS

Primary geosynthetic reinforcement must consist of a geogrid meeting the requirements of Publication 408, Sections 738.1 and 738.2, Class 1, Type A or B. All primary, secondary and wrap geosynthetics are to be designed and specified for 100 percent coverage for each layer of geosynthetic. No breaks in coverage are permitted in any layer for any design section. Keep the number of primary geosynthetic strength and length configurations in any one vertical design section to a minimum. Multiple configurations complicate construction of a slope, and any savings realized in geosynthetic materials is lost or exceeded in the difficulty and costs associated with the complex design. In most cases a single strength/length configuration is preferable for any one design section. Multiple configurations may be considered for high slopes (i.e., greater than 30 feet) or special conditions, but be kept to as few as practical. Keep the length of design sections as long as practical, but no less than 200 feet unless subsurface conditions or reinforced slope geometry are highly variable, or other complex conditions exist.

11.8.1 Vertical Spacing

The maximum vertical spacing for primary geosynthetic, S_{vp} , is 18 inches. At the top of the GRS slope, provide an additional layer of primary geosynthetic at subgrade elevation when the subgrade elevation is greater than or equal to nine inches above the previous layer of primary geosynthetic. The maximum vertical spacing for secondary geosynthetic, S_{vs} , is six inches. Refer to Standard Drawing RC-14 for details.

11.8.2 Primary Length

For each design section, determine the zone to be reinforced by examining the full range of potential failure surfaces and failure modes by analyzing the proposed slope in an unreinforced condition using the latest version of SLIDE7. Consider both circular arc and sliding wedge type failures. Examine possible failures through the toe and face of the slope, and compound and deep seated failures extending into the foundation.

The location of surfaces just meeting the target minimum required factors of safety indicated in [Table 11.6.3-1](#) for the various modes of failure, excluding external horizontal sliding, defines the approximate zone to be reinforced. These modes of failure include internal stability and failure surfaces extending below the toe of the slope. Surfaces extending below the toe indicate compound or deep stability problems and/or foundation bearing capacity problems that must be addressed as part of the design.

The length determined from these analyses shall be used as the preliminary primary geosynthetic length (L_P) for the full height of the zone, as shown in [Figure 11.2-1](#). The minimum length of primary geosynthetic is eight feet. The final primary geosynthetic length is determined per Sections [11.8.3](#) and [11.8.5](#).

11.8.3 Primary Tensile Strength

Use SLIDE7 to determine the primary geosynthetic design tensile strength. Use the preliminary primary tensile length (L_P) determined in [Section 11.8.2](#) in stability analysis of the final design condition. Include all temporary loading conditions and surcharge loads. Consider both circular arc and sliding wedge type failures, and examine possible failures through the toe and face of the slope, and compound and deep seated failures extending into the slope foundation.

Use SLIDE7 to optimize the primary geosynthetic length and tensile strength requirements. For multiple surface search routines, start with broad initiation and termination limits, and tighten limits on iterative runs to optimize the design.

Once required lengths and tensile strengths appear to be optimized, check in front of and through the leading edge of the reinforced zone with migrating focused initiation and termination zones using a high density of surface generation, to assure no problem areas exist. Conduct the same analyses through the reinforced zone in limited search areas systematically moving across the slope until the trailing edge and outside the limits of geosynthetic have been similarly

checked to assure that no problem areas exist. Adjust primary geosynthetic lengths and/or strengths as necessary.

Use segment lengths that provide failure surfaces of at least 15 slices for small, non-complex slopes, and 20 to 25 slices for larger or complex conditions (e.g., highly variable surface geometries or subsurface conditions). Do not exceed 10 feet segment lengths for any slope unless the analysis is producing more than 30 slices. Some slope and loading configurations may produce more than one critical zone of stability. All critical areas must be accounted for in the design.

These analyses determine the design tensile strength (T_D) and design length (L_D) for the primary geosynthetic.

11.8.4 Required Geosynthetic Tensile Strength

Once the optimal primary geosynthetic strength, T_D , and length, L_D , combination have been determined for a given design section, determine the required geosynthetic tensile strength for construction (T_{10}) as indicated below:

$$T_{10} = T_p RF_{OV}$$

where,

T_{10} = Primary geosynthetic tensile strength, determined at 10 percent strain, required for construction, (kips/ft.)

T_D = Primary geosynthetic design strength as determined above, (kips/ft.)

RF_{OV} = Overall material reduction factor for the primary geosynthetic accounting for creep, installation damage, degradation, and polymer type (dimensionless). See Publication 408, Section 738.2 for both Type A and Type B, Class 1 Geogrids, for required RF_{OV} by geogrid polymer type.

Calculate required T_{10} for both Class 1, Type A - PET and Class 1, Type B - HDPE polymers. State the requirements for both Class 1, Type A and B geogrids in the Department's Standard Special Provision cXXXXX ITEM 9200-0200 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE A; ITEM 9200-0201 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE B; ITEM 9200-0202 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE C, ITEM 9200-0203 GEOSYNTHETIC REINFORCED SOIL SLOPE, TYPE D; ITEM 9200-0205 EXCAVATION OF UNSUITABLE FOUNDATION MATERIALS.

11.8.5 Check Primary Geosynthetic Length for Sliding Stability

Once the primary geosynthetic length (L_D) has been determined per [Section 11.8.3](#), sliding stability must be checked as indicated below. If the factor of safety for sliding (FS_S) obtained with this length meets the minimum required in [Table 11.6.3-1](#), then use this value of L_D as the final design value for the design section being investigated. If the factor of safety does not meet minimum requirements, then increase the primary geosynthetic length until the required

minimum FS_S is obtained. If an increase in length is required, the geosynthetic length required to meet the minimum factor of safety for sliding becomes the new design length (L_D) for the design section.

Determine the sliding resistance of the reinforced mass for all zones as follows:

$$FS_S = \frac{\text{Resisting Force}}{\text{Sliding Force}} = \frac{(W + P_a \sin \phi_b) \tan \phi_{sg}}{P_a \cos \phi_b}$$

$$\text{where, } W = \frac{1}{2} L^2 \gamma_{rf} \tan \beta \quad (\text{for } L \tan \beta \leq H)$$

$$W = (LH - \left(\frac{H^2}{2 \tan \beta}\right)) \gamma_{rf} \quad (\text{for } L \tan \beta > H)$$

$$P_a = \frac{1}{2} \gamma_{rm} H^2 K_a$$

and where,

W = weight of reinforced mass in and above zone, kips;

L = length of bottom reinforcing layer in each zone where there is a geosynthetic length change, ft.

H = height of slope including zones above, ft.

FS_S = factor of safety for sliding (≥ 1.5 or 2.0 , as required, see [Table 11.6.3-1](#))

P_a = force due to active earth pressure including zones above, kips

ϕ_b = friction angle of reinforced fill, degrees

ϕ_{sg} = soil-geosynthetic direct shear resistance (determined in accordance with ASTM D 5321), or geosynthetic-to-geosynthetic direct shear resistance where applicable. If fill/geosynthetic specific information is not available, use 0.67 of design friction angle ($0.67\phi_b$) of reinforced fill for geotextiles and 0.85 of design friction angle ($0.85\phi_b$) of reinforced fill for geogrids

β = slope angle, degrees

γ_{rf} = unit weight of reinforced fill, kips/ft³

γ_{rm} = unit weight of retained soil mass or backfill, kips/ft³

11.8.6 Secondary Geosynthetic

All reinforced slopes require a secondary geosynthetic consisting of a Class 4, Type A geotextile, meeting the requirements of Publication 408, Section 735. Class 4, Type A geotextile consists of a non-woven needle punched fabric having a visual appearance of a thick felt. The secondary geosynthetic has multiple functions, including:

- edge reinforcement to permit proper compaction along the face of the slope
- internal drainage of the slope in the plane of the fabric
- control for lift thickness
- reinforcement and minimize stresses at the face of the slope.

All layers of secondary geosynthetic must extend from the face of the wire mesh forms for Type A and Type B slopes, from the face of the geosynthetic wraps for Type C slopes, and from the back of the gabion baskets for Type D slopes, to a minimum of the full length of the longer layer of primary geosynthetic directly above or below the layer of secondary geosynthetic. For slopes where inundation by water is a concern, or that are to be constructed against existing slopes that exhibit significant seepage, the need for additional portions of secondary geosynthetic to extend additional length, or a supplementary drainage system behind and/or beneath the slope must be evaluated and specified as necessary as discussed in [Section 11.3](#).

11.8.7 Face Wrap or Backing

For Type B, C, and D slope face types, the geosynthetic face wrap or backing may serve a number of functions depending upon slope type, including: stabilize the fill at the face of the slope, prevent piping and erosion, and function as a filter. It is placed in addition to the secondary geosynthetic. The geosynthetic face wrap or backing is not intended to impart any reinforcement function to the reinforced soil system, and should not be accounted for in the design. The face wrap or backing geosynthetic shall be specified as a Class 4, Type A geotextile.

11.9 CONSTRUCTION RELATED CONSIDERATIONS

11.9.1 Placement of Geosynthetics

GRS slopes must be constructed in level horizontal layers that do not follow existing or proposed grades. Specify the placement of all primary and secondary geosynthetics horizontal and level, both in the direction of reinforcement and the direction of the roadway.

11.9.2 Benching

When reinforced slopes are being constructed against existing natural or fill slopes, benching of the existing slope is required to tie the proposed and existing slopes together. If a standard or other project specific detail is not specified, use the default benching detail as indicated in Standard Drawing, RC-14.

11.9.3 Temporary and Construction Conditions

In design of the reinforced slope, consider both temporary loading conditions during construction, and construction sequencing in addition to final conditions. Maintain the required factors of safety for temporary construction conditions, unless other specific values are approved by the District Geotechnical Engineer (DGE).

11.9.4 Transitions for Slopes with Surface Grades

While reinforced slopes are constructed horizontal and level, existing ground surfaces and finished top of slope lines are at various grades. Require foundations for reinforced slopes to be prepared horizontal and level, with embedment as necessary to provide required internal and external stability. Clearing and grubbing must be conducted according to Publication 408, Section 201. Foundation elevations shall be stepped in increments or multiples equivalent to the spacing of the primary geosynthetic.

At the top of the slope, both primary and secondary geosynthetics shall extend laterally until they daylight at the subgrade elevation, or at the indicated plan elevation. Subgrade elevation is defined as either the layer immediately beneath the subbase for pavements, or immediately beneath topsoil. For Type A and B slope face types, wire mesh forms shall be trimmed to follow the finished grade.

For Type C slopes, top of geotextile wraps shall be parallel to finished grade, ending at the subgrade elevation. For Type D slope face type, reduced height gabions shall be used for the top of the reinforced slope, following the finished grade as close as possible. The reduced height gabions shall still be placed level and horizontal, stepping the gabions as necessary to closely follow finished grade.

11.10 DESIGN SUBMISSION

Submit the GRS slope design as part of the Geotechnical Engineering Report unless required by the DGE to be submitted under separate cover for timelier project completion. The submission shall include:

- typical cross sections for each design section indicating: GRS slope type(s), primary geosynthetic types and length, existing and proposed grades, foundation and existing slope conditions and associated parameters
- design parameters and required properties for the reinforced fill and primary geosynthetic
- calculations and analyses for all design requirements including internal and external stability, and temporary construction conditions
- any necessary or special drainage treatments and details
- any other required details

A data table must be included for each design section that indicates:

- State Route (SR)
- station limits and offset (right or left of baseline) of the design section
- length(s) and required tensile strength(s) of primary geosynthetic
- slope angle
- required retained fill properties
- number of layers of primary geosynthetic (for each geosynthetic type in the section)

- reinforced slope height
- location of the geosynthetic when multiple strengths or lengths are used for the primary geosynthetic in a design section.

See [Table 11.10-1](#) for the proper configuration of the table with sample data.

Table 11.10-1 – GRS Slope Summary Table Requirements Example

Primary Geosynthetic Schedule										
Required Geosynthetic and Reinforced Fill Parameters										
SR	Station Limits	L/R	Slope (H:V)	Slope Height (ft.)	Layer Ultimate Tensile Strength (lbs./ft.)	Length (ft.)	No. Layers	Location	Unit Weight Range (pcf)	Min. Friction Angle (degrees)
59	25+50 to 28+00	L	0.5:1	30	10,000	20	10	Bottom Half	110-130	30
					6,000	25	10	Top Half		
	28+00 to 31+00	L	1:1	35	6,000	25	24	Entire Section	110-130	30
167	385+00 to 389+00	R	0.8:1	25	6,000	20	18	Entire Section	110-130	32

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2018
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 12 – GEOCELL

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12.1 INTRODUCTION

Geocell was developed by the U.S. Army Corps of Engineers, Waterways Experiment Station, in the mid-1970's to achieve a method of rapid construction of sand roads for beach landings and desert operations. Materials in these settings typically consist of dry, fine and uniformly graded sand that is not capable of supporting repetitive wheel loading without significant rutting. The rutting is due to localized shear failure of the near surface material. As an alternative to stabilization with chemical additives requiring mixing and curing time, three-dimensional cellular confinement of the on-site loose sands was determined to be practical, very cost effective and highly efficient. Based on research performed by the U.S. Army Corps of Engineers, proprietary geocell systems were developed for commercial use to increase the load distribution capacity of unbound aggregates over soft soils.

Geocell is a generic term for polymer grid cell reinforcement. It is known by a variety of trade names and consists of polymer, typically high density polyethylene (HDPE), sheet strips joined by a series of full-depth ultrasonic welded seams. The welded seams are aligned perpendicular to the longitudinal axis of the strips and are offset on adjacent strips to form a cellular array when expanded. The geocell is stored and shipped in its collapsed form, making handling of the material relatively easy. When expanded during installation the welded strips form the walls of a flexible, three-dimensional, cellular confinement structure into which specified granular fill, usually No. 2A coarse aggregate, is placed and compacted. When filled, geocell results in a system that restricts lateral movement of the infill through confinement by the cell walls. The result is a highly efficient load distribution system that significantly reduces the stress transferred to the underlying soil subgrade. Various cell depths and effective cell diameters are available for application with various load and foundation/subgrade strength conditions. Geocell is available with smooth cell walls, textured cell walls to provide improved frictional resistance between the walls and the infill, and perforated walls to provide improved frictional resistance and/or lateral drainage through the geocell.

12.1.1 Purpose

The purpose of this chapter is to address the use of geocell in subgrade stabilization applications where soft subgrades are anticipated based on design activities or are identified during construction. It is acknowledged that there are other uses for geocell, such as erosion protection (slopes and channels), steep slopes, retaining walls/hybrid walls, and pervious pavements. These uses are not covered in these guidelines. Applications other than subgrade stabilization must be designed and constructed according to other Department standards and/or manufacturer's recommendations, as appropriate.

12.1.2 Function and Use

Geocell filled with granular material, typically No. 2A coarse aggregate, is a proven, effective method for carrying wheel loads over very soft subgrade soil. This is especially true for localized areas of unstable, fine-grained soils identified during design or encountered during construction. The geocell provides horizontal confinement for the aggregate infill, resulting in

the creation of a relatively rigid platform to spread the applied load and reduce stresses on the soft subgrade.

Geocell is most effective for soils with very low California Bearing Ratio (CBR) values, normally less than or equal to three. As the CBR value of the subgrade increases, there is less need for the load-spreading capability that geocell provides. In such situations, it may be more appropriate and cost effective to incorporate a biaxial geogrid in the subbase layer, or simply increase the subbase thickness to provide the necessary support. To assess the value of using geocell over a soft subgrade, the cost of the geocell materials and placement has to be compared to the material and placement cost of the additional aggregate subbase necessary for the subgrade strength and conditions. An economic analysis needs to be performed to determine if the use of geocell is warranted for the specific site conditions.

12.1.3 Design Background and Applicability of Use

Typically, geocell manufacturers and/or distributors provide design methods for calculating the required geocell thickness, or the resulting reduction in aggregate thickness when used in conjunction with paved surfaces. However, as with other geosynthetic materials, no widely-accepted design method currently exists to account for the structural benefit of the geocell that is consistent with the pavement design methodology used by the Department. Structural coefficients of various standard, conventional paving courses and aggregates have been developed based on roadway performance data that has been collected over significant periods of time. Emerging technologies for stabilizing soft subgrades have not been frequently constructed and monitored to obtain the appropriate data to define structural coefficients for these materials. The variability of proprietary materials and differing site conditions also restrict the ability to generalize the structural benefit of such systems. For these reasons, the Department is not able to account for the structural benefit from the geocell in determining the required pavement section. However, the benefit of the geocell is recognized in the demonstrated ability to carry loads over very soft subgrade soils, and for construction conditions where other methods of subgrade treatment are not functionally or economically feasible.

Despite the lack of a strict design method, geocell has been used successfully by the Department to carry loads over localized, very weak subgrades, and may be an effective option in similar situations. This is especially true in cases where construction is underway and either local areas of weak subgrade are identified by the contractor's operations or adverse weather conditions cause an otherwise adequate subgrade to become unstable.

Situations like these have traditionally been handled by undercutting significant portions of the weak or unstable subgrade, lining the undercut with geotextile, and then replacing the excavated material with either borrow rock from on-site sources, or processed aggregate from off-site sources. The undercuts can become quite extensive and costly, disrupt construction activities or schedules, or may not be practical because of underground utilities. Also, this practice could result in creating a "bath-tub" effect where water collects under the pavement creating long-term stability and durability problems for the pavement. Thus, the use of geocell in these situations could result in significantly less excavation and backfill, save time in

mitigating a weak or unstable subgrade, possibly eliminate or reduce contractor delays, and potentially provide cost savings to the Department.

12.2 GEOCELL SELECTION GUIDELINES

Publication 408, Section 737 offers three different types of geocell based on the nominal cell area: Type A (44.8 in²), Type B (71.3 in²), and Type C (187.0 in²). Standard available geocell depths range from three inches to eight inches for each geocell type. The required type and depth of the geocell will be controlled by a variety of factors, including subgrade strength, wheel loads, geocell wall characteristics (e.g., smooth, textured, or perforated), infill material, and cell size (effective diameter). Even though no widely accepted standard design method exists for determining the required depth of geocell under paved roadways, the following guidelines may be used to select an appropriate geocell to stabilize an unsuitable subgrade:

1. The use of geocell is most applicable for subgrades with CBR values less than or equal to three. The need for the load spreading capability provided by geocell is greatly reduced as the strength of the subgrade increases. Also, note that Publication 242 does not allow pavements to be constructed on subgrades with CBR values less than five without providing stabilization to increase the CBR to an acceptable level. For subgrades with CBR values greater than three, it may be more appropriate and cost effective to increase the aggregate thickness and/or use a geogrid in the subbase layer rather than installing geocell.
2. The required geocell depth generally increases as the CBR decreases.
3. The required geocell depth increases as the applied wheel load increases or as the number of design load-cycles increases.
4. The geocell wall characteristics generally have some effect on the required depth. Textured and perforated cell walls typically allow for slightly less geocell depth than smooth cell walls. Perforated cell walls are most efficient due to particle interlock between aggregates in adjacent cells.
5. Geocell with perforated cell walls should always be used when incorporated in pavement subbase to assure proper lateral drainage and for any other application where the movement of water laterally through the geocell is necessary or desired.
6. Cell size has a significant effect on the required geocell depth. Where the loaded area is relatively small, such as from vehicle wheel loads, cells with a smaller effective area are most efficient. While in theory it may be possible to demonstrate that a deeper Type C geocell is as efficient as a shallower Type A or B geocell, in practice only Types A and B geocells should be used for subgrade stabilization applications. The effective area of a Type C geocell is so much larger than the footprint of the wheel load that long-term stress reduction cannot be assured. The use of Type C geocell should be reserved for erosion control,

unpaved low-volume roads with adequate subgrade strength (i.e., to hold aggregate in place), and similar applications.

7. Geocell is most effective when placed as close as possible to the applied load. For use as subgrade support during construction and under pavement, geocell should typically be placed with a minimum of one-inch of aggregate surface above the geocell to protect it from construction traffic and heat during paving. If the required thickness of subbase based on the pavement design is greater than the depth of the geocell, place the additional required thickness of subbase below the geocell.
8. Place a Class 4, Type A geotextile under geocell placed directly on fine-grained subgrade or under subbase placed below the geocell to prevent fines migrating into and contaminating the geocell infill (i.e., subbase).
9. Aggregate infill material should be No. 2A coarse aggregate. Clean crushed stone infill, such as AASHTO No. 57 coarse aggregate is less stable and less efficient as a geocell infill material than No. 2A. AASHTO No. 57 coarse aggregate should be avoided when using geocell in a subgrade stabilization application unless project specific needs dictate its use.

A typical section showing a geocell application is provided in [Figure 12.2-1](#).

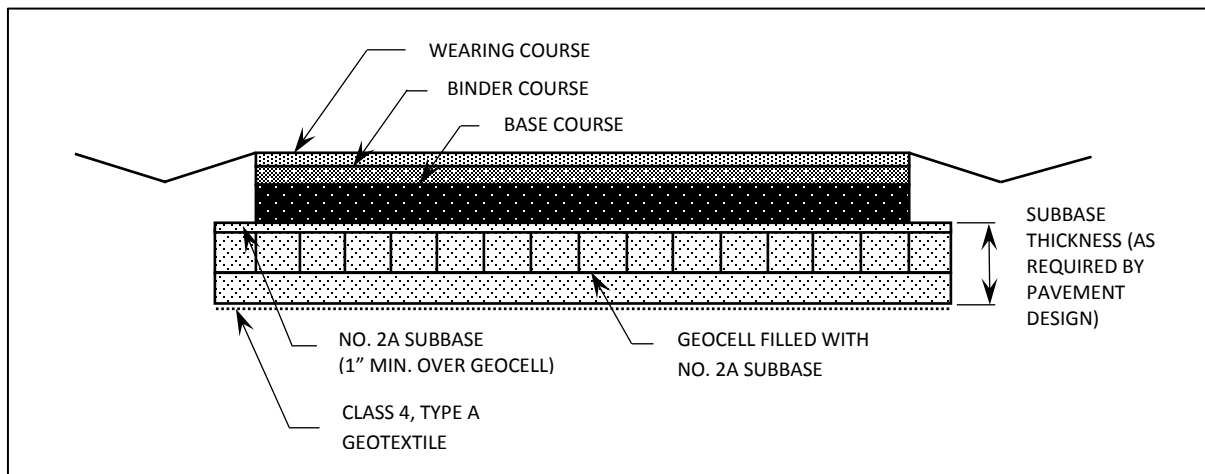


Figure 12.2-1 – Typical Section

As previously discussed, the structural benefit of geocell cannot be directly accounted for when designing pavements with the methods that the Department currently uses. However, a design method exists for determining the required depth of geocell for unpaved roads. Considering that significant construction traffic typically travels on the subgrade and subbase before paving is completed, it is logical to design for this condition when soft subgrade soils are present and require stabilization.

[Table 12.2-1](#) presents the results of calculations performed to determine the required depth of geocell for subgrade CBR values ranging from less than one to five. Assumptions regarding the depth of subbase above the geocell, tire contact pressure, wheel load, geocell wall characteristics, and type of infill are noted below the table. The assumed parameters are believed to be representative of conditions commonly encountered during construction. The table indicates acceptable/efficient geocell types and depths for the various CBR values investigated and the assumed parameters. [Table 12.2-1](#) can be used as a guide to select an appropriate geocell to stabilize soft subgrades encountered during construction. If project specific parameters vary from the assumed parameters used to develop the table, additional design calculations will be required.

Table 12.2-1 – Recommended Geocell for Subgrade Stabilization

CBR	Geocell Type											
	Type A				Type B				Type C			
	Geocell Depth (in)				Geocell Depth (in)				Geocell Depth (in)			
	3	4	6	8	3	4	6	8	3	4	6	8
≤1				X				X				
2			X			X						
3			X			X						
4		X				X						
5	X				X							

- Notes: 1. “X” indicates recommended for assumed parameters
2. Assumed parameters:
 Depth of subbase above geocell = 1 inch
 Tire contact pressure = 100 pounds per square inch
 Wheel load = 6,000 pounds
 Geocell walls = Perforated
 Geocell infill = Compacted No. 2A coarse aggregate

As with any method of addressing unstable subgrade engineering judgment is required when selecting geocell depth and cell size for stabilizing unsuitable subgrades. In general, geocell with depths on the order of six to eight inches and nominal cell areas of less than about 72 in² (i.e., Types A and B geocell) are appropriate for stabilizing areas of unsuitable subgrade with CBR values less than or equal to three, when significant construction equipment and traffic will be traveling on the geocell reinforced subbase prior to paving. Type C (i.e., nominal cell area of 187 in²) geocell should not be used for stabilizing soft subgrades subjected to wheel loads. The use of shallower depth geocells should be limited to areas where minimal construction equipment and traffic will travel on the geocell reinforced subbase prior to paving, or where the unstable subgrade is marginally acceptable in an unreinforced condition.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

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GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 13 – GEOFOAM

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13.1 INTRODUCTION

This chapter of this publication provides guidelines for the use of geofoam on Department roadway and structure projects. Included are a discussion of the typical properties of geofoam, applications where geofoam may be beneficial, variables that affect the cost of using EPS geofoam, and geofoam design and construction requirements and considerations. This chapter also provides a profile and section view of a geofoam embankment, details for use on Department projects, and a sample calculation for determining the compressive resistance requirements of the geofoam for the anticipated loading.

Geofoam can be used to construct embankments and backfill behind structures. Geofoam is a generic term used to describe any foam material that is used for a geotechnical application. Geofoam is an extremely lightweight, manufactured material produced by an expansion process that yields numerous closed, gas filled cells. A “blowing agent gas”, which is typically something other than air, fills the cells during the manufacturing process. Air eventually displaces the blowing agent gas after the geofoam is manufactured. Most geofoam materials are polymeric (i.e., plastic), and the most commonly used polymer to make geofoam is polystyrene.

13.2 OVERVIEW OF GEOFOAM MANUFACTURING PROCESS

There are currently two methods used to manufacture polystyrene foam, and they are the molding method and the extrusion method.

1. The molding method is a two-step process where, in the first step, tiny polystyrene beads are expanded to make “pre-puff” and, in the second step, the pre-puff is fused together in a molding process. This white material is typically referred to as Expanded Polystyrene (EPS), and the individual beads can generally be seen.
2. The extrusion method is a single, continuous process that produces expanded polystyrene foam referred to as Extruded Polystyrene (XPS). XPS foam is typically colored (e.g., blue, pink, etc.) during the manufacturing process for proprietary purposes only.

Note that the term “styrofoam” is commonly and erroneously used as a generic term, when in actuality this term is a registered trademark for a specific brand of XPS foam. Consequently, the term “styrofoam” should not be used when referring to EPS or XPS foam in general terms.

Polystyrene foam is manufactured in a variety of shapes and sizes depending upon the method and equipment used by individual manufacturers. EPS foam is usually molded in “block” form, and a typical block can measure 2 feet high by 4 feet wide by 16 feet long. The molded blocks are then cut in the factory to the size required for the specific application. XPS foam is typically extruded in “sheets” no thicker than 4 inches. **Since the block form of foam is more convenient for large volume applications compared to thin sheets, EPS foam is almost**

exclusively used on roadway and structure projects. The remainder of this chapter will concentrate on EPS geofoam.

13.3 EXPANDED POLYSTYRENE (EPS) GEOFOAM PROPERTIES

The physical properties of EPS geofoam are varied and controlled during the manufacturing process, and these properties are specified in ASTM D6817, “Standard Specification for Rigid Cellular Polystyrene Geofoam”. Seven types or designations of EPS geofoam are listed in D6817, including: EPS12, EPS15, EPS19, EPS22, EPS29, EPS39 and EPS46. Note that the EPS number designation corresponds to the required density of the geofoam in kilograms per cubic meter (kg/m³). For example, the minimum required density of EPS22 is 21.6 kg/m³.

Table 13.3-1 – Physical Property Requirements of EPS Geofoam^{1, 2}

Property	Test Method	ASTM Designation						
		EPS12	EPS15	EPS19	EPS22	EPS29	EPS39	EPS46
Density (pcf)	ASTM D1622	0.7	0.9	1.15	1.35	1.80	2.4	2.85
Compressive Resistance at 1% Deformation (psi)	ASTM D1621	2.2	3.6	5.8	7.3	10.9	15.0	18.6
Compressive Resistance at 5% Deformation (psi)	ASTM D1621	5.1	8.0	13.1	16.7	24.7	35.0	43.5
Compressive Resistance at 10% Deformation (psi)	ASTM D1621	5.8	10.2	16	19.6	29.0	40.0	50.0
Flexural Strength (psi)	ASTM C203	10.0	25.0	30.0	35.0	50.0	60.0	75.0
Oxygen Index (Vol. %)	ASTM D2863	24.0	24.0	24.0	24.0	24.0	24.0	24.0
Water Absorption (Vol. %, max)	ASTM C272	4	4	3	3	2	2	2

Notes: 1. All values are minimum unless indicated otherwise.

2. Reference: ASTM D6817, Table 1

The two properties of geofoam that are of particular interest for geotechnical applications and that are listed in D6817 are the minimum density and the minimum compressive resistance. Based on the requirements of D6817 for the seven EPS geofoam types, the minimum density varies from 0.7 to 2.85 pounds per cubic foot (pcf), and the minimum compressive resistance varies from 2.2 to 18.6 pounds per square inch (psi), as measured at 1% strain/deformation. Due to the very small variation in density between the seven different types of EPS geofoam, and since this variation will generally be negligible with respect to geotechnical and structural analyses, the compressive resistance will typically control the selection of the geofoam designation required for the specific project application. As discussed in [Section 13.6.3.4](#), geofoam types EPS22 and EPS39 will be most commonly used on Department projects. Note

that Publication 408, Section 219, Geofoam Lightweight Fill, refers to EPS22 as Type 1 geofoam and EPS39 as Type 2 geofoam.

Note that only geofoam products included in Bulletin 15 can be used unless project specific approval is obtained to use another manufacturer or EPS designation.

13.4 TYPICAL USES OF EPS GEOFOAM

EPS geofoam used on roadway and structure projects typically weighs less than 2 lbs/ft³. In comparison to other materials, EPS geofoam weighs approximately:

- One percent to two percent of the weight of typical earth materials (i.e., soil and rock);
- Three percent of the weight of typical lightweight aggregates;
- Less than 10 percent of foamed/cellular concrete.

Since EPS geofoam is extremely lightweight, it is typically used on roadway and structure projects for control of settlement, mitigation of slope instability, and reduction of horizontal and vertical loads on structures.

13.4.1 Settlement Control

When the magnitude and/or time rate of settlement estimated from the load of traditional embankment material is not tolerable, measures must be taken to control settlement either by reducing settlement and/or decreasing the time needed for settlement to occur. These measures often include one or more of the following:

- wick/prefabricated vertical drains or sand columns,
- preload or surcharge embankment,
- ground improvement – Deep Dynamic Compaction (DDC), stone/aggregate columns, jet grouting, etc.

In lieu of these measures, EPS geofoam can be used as the fill material to construct embankments to significantly reduce, or completely eliminate, settlement.

Long-term (i.e., secondary consolidation) settlement of organic soils cannot be mitigated using conventional measures typically used for primary consolidation (e.g., wick drains, preload embankment, DDC, etc.). Therefore, the extremely light weight of EPS geofoam can be particularly useful to eliminate or significantly reduce secondary consolidation settlement when such settlement is anticipated and the expected magnitude over the life of the facility will potentially pose a problem.

Similarly, EPS geofoam may be useful in minimizing or eliminating differential settlement. For instance, if an embankment or structure is proposed to be widened, and settlement from traditional fill materials are estimated to create differential settlement that is not tolerable, or settlement that may be detrimental to existing pavement or structures, the use of

EPS geofoam may be a viable option. Additionally, where intolerable settlement of an approach embankment to a bridge that is founded on a “rigid”, deep foundation is anticipated, the use of EPS geofoam may be beneficial to eliminate or reduce the settlement to a tolerable amount.

Embankments constructed over existing utilities, archeological areas or other sensitive features could also be constructed with EPS geofoam to reduce the load and corresponding settlement, and minimize the impact of embankment construction. If used over utilities, it would be important that the EPS geofoam system (including membrane barriers) be replaced as per original design should any work be conducted on utilities that requires removal of the geofoam system.

13.4.2 Slope Stability

EPS geofoam can also be considered for fill material where global slope stability is a concern, either for new embankment construction or for remediation of a slope failure/landslide. The EPS geofoam blocks are extremely light weight and add insignificant loads, and therefore are advantageous when constructing embankments on unstable/low shear strength material. Replacing existing soil/rock embankment material with EPS geofoam (i.e., load reduction) may be sufficient to remediate an active landslide when used to reduce the driving force along the failure plane.

13.4.3 Structure Backfill

Under certain conditions, EPS geofoam is advantageous for backfilling behind structures. The light weight of the EPS geofoam applies insignificant vertical load to structure foundations. Additionally, due to the light weight of the geofoam and since geofoam blocks are inherently stable when stacked vertically, virtually no lateral load is applied to bridge abutments and retaining walls from EPS geofoam when used as backfill. In such situations, either the EPS geofoam must extend beyond the limits of the active earth pressure wedge, or lateral loads from fill placed behind the EPS geofoam must be accounted for in the design. Geofoam can be used for structure backfill on new structures or where the height of an existing structure must be increased but the existing foundation cannot support the load from traditional structure backfill material.

13.5 ECONOMIC ANALYSES OF EPS GEOFOAM

Although EPS geofoam has some unique and beneficial qualities that other materials do not have, in most cases there will be other more economical options that will accommodate project needs. For example, in lieu of using EPS geofoam to limit settlement, lightweight foamed concrete may be a viable option, or a surcharge embankment, with or without wick drains and a quarantine period may be an option. Alternatively, a ground improvement or modification technique may be used to either increase the strength of the foundation soil or transfer loads to a deeper, more competent stratum. As in any design situation, where multiple feasible options exist to address the site conditions and project needs, an economic analysis should be performed and considered to help select the most viable alternative.

Similar to other construction materials and alternatives, there are multiple factors that must be considered when estimating the cost of EPS geofoam for use on a roadway or structure project. Below are some, but not necessarily all, of the factors that must be taken into account when considering the cost of EPS geofoam for a specific project. Cost data is presented later in this section. The cost data presented herein is for information purposes only, and should not be used for estimating the cost of EPS geofoam for a specific project unless the information is verified. When preparing a cost estimate for a specific project, manufacturers and suppliers of EPS geofoam should be contacted to obtain current price information, and bid price data from recent projects that included EPS geofoam can also be considered.

13.5.1 EPS Geofoam Cost Versus Density

The cost of EPS geofoam is directly related to the density of the geofoam. The density of the geofoam is controlled by the amount of polystyrene used, and the higher the geofoam density the more polystyrene is needed. Therefore, the material cost of the EPS geofoam increases as the density increases. Below are costs for various types of EPS geofoam that were obtained from a manufacturer in mid-2016:

EPS19 = \$60/cy
EPS22 = \$70/cy
EPS29 = \$90/cy
EPS39 = \$125/cy

These prices assume a quantity of at least 10,000 cubic yards will be used, include an approximately \$3.50/cy transportation cost for shipping within 200 miles of the factory, but do not include the cost of placing the material. Due to the considerable cost difference between the various geofoam densities, it is important to determine the appropriate density for the intended application.

13.5.2 EPS Geofoam Cost Versus Crude Oil Price

As previously indicated, the most commonly used polymer to make EPS geofoam is polystyrene. Polystyrene consists of long chains of the styrene monomer or molecule, and the raw materials used to make styrene are obtained from crude oil. The cost of the raw materials (i.e., styrene) used to make EPS geofoam have varied with the cost of crude oil in the past. Information obtained from an EPS manufacturer's website in early 2013 and as indicated in [NCHRP Project No. 24-11\(02\) Final Report](#), the price of EPS geofoam disproportionately increases with higher geofoam density as crude oil price rises. However, more recent information indicates that as the use and manufacturing facilities of EPS geofoam have continued to grow, it has become more of an independent market that doesn't as closely follow the trends of petroleum prices.

13.5.3 Transportation Cost

The cost of transporting the EPS geofoam from the manufacturer to the project site must be considered. Similar to any material, the longer the distance of the haul the more expensive

the material will be to use. As discussed above, one estimate from a manufacturer indicated transportation cost at approximately \$3.50 per cubic yard for a project site located within 200 miles of the manufacturing facility. Based on other available shipping cost information, this price is probably more reasonable for a shorter hauling distance of approximately 100 miles to the site. Since the geofoam is extremely lightweight, shipping constraints are based on volume and not weight. Typically, approximately 125 cubic yards of material can be transported in a single load.

13.5.4 Placement Cost

When estimating the cost of EPS geofoam, placement of the blocks must be accounted for in addition to material and transportation costs. Information included in the [NCHRP Web Document 65](#) indicates placement cost can vary from \$10 to \$25 per cubic yard. These costs were from projects constructed more than a decade ago (i.e., pre-2003); therefore, more up to date costs should be obtained when estimating EPS geofoam costs for a specific project. Placement cost will depend on numerous variables, including total volume of material placed, experience of contractor, complexity of the geometry and amount of specialty fabrication/cutting required.

13.5.5 Hydrocarbon Resistant Geomembrane Cost

EPS geofoam is not resistant to petroleum products, including gasoline and diesel fuel, as well as other solvents. Consequently, when EPS geofoam is used on Department projects, a hydrocarbon resistant geomembrane must be used to encapsulate the geofoam as discussed in [Section 13.6.6](#). The geomembrane must be accounted for in the cost estimate. Hydrocarbon resistant geomembranes are more costly to manufacture than conventional geomembranes used for water containment and municipal landfills. Available data in 2014 indicates that the material cost alone can be approximately \$10 per square yard. Therefore, the cost of the hydrocarbon resistant geomembrane will be a significant part of the overall EPS geofoam system. Some situations may require the use of a concrete distribution slab constructed on top of the geofoam. If a concrete slab is required by design, its cost must also be accounted for in the EPS geofoam system.

13.5.6 Miscellaneous Costs

Other miscellaneous costs that must be considered when estimating the cost of EPS geofoam for use as lightweight fill include: bedding material, mechanical connectors, insecticide and any non-standard, project specific requirements. Although these costs are typically minimal compared to cost factors discussed above, they should none the less be included in a cost analysis or estimate.

13.6 DESIGN REQUIREMENTS AND CONSIDERATIONS

Geotechnical analyses are required to design EPS geofoam embankments. At a minimum settlement and global slope stability analyses must be performed. In some cases, additional analyses including bearing resistance, sliding and seismic stability must be performed. These

analyses are discussed below along with other design requirements and considerations for the use of EPS geofoam on Department projects. Additional details and discussion concerning analyses associated with EPS geofoam embankments can be found in the NCHRP Web Document 65.

13.6.1 Settlement Analyses

In most cases, EPS geofoam will be used, at least in part, to eliminate or reduce the estimated settlement from embankment construction to a tolerable level. Settlement analyses must be performed to not only justify the need for the use of EPS geofoam in place of more standard and economical fill materials (i.e., embankment material or structure backfill), but to also determine the required limits of the EPS geofoam within the embankment cross-section. Immediate, consolidation, and secondary consolidation settlement must be considered.

Once it is determined that EPS geofoam is needed and is a viable alternative, the settlement analysis will indicate one of several scenarios with respect to placement limits. These may include constructing the entire fill section using EPS geofoam (with exception of required embankment cover), constructing a limited zone of the fill using EPS geofoam (this could include specific vertical limits of the fill or a specific horizontal area where load reduction is required), or some combination of the two.

Since EPS geofoam is of substantially higher cost than standard embankment and structural fill materials, it may be very important to limit the amount of EPS geofoam in the embankment cross-section. Where settlement analyses indicate it is acceptable and when practical, construct the lower portion of the embankment cross-section with standard materials, and place EPS geofoam above the embankment/structure backfill material to complete construction of the embankment. Conversely, settlement analyses may indicate that the load from the embankment cover material and pavement section above the EPS geofoam embankment results in unacceptable settlement. In these cases, it may be necessary to remove in-situ material below the proposed embankment and replace it with EPS geofoam in order to balance the load from the cover material and pavement section. Since EPS geofoam has a density much less than that of water it is important that the local groundwater elevation be identified, so as to avoid problems with buoyancy as discussed in [Section 13.6.4.3](#).

13.6.2 Global Slope Stability Analyses

EPS geofoam is often used to address settlement concerns; therefore, the in-situ soil that the geofoam is placed on is often soft, saturated and low strength. It is important that global slope stability be considered when designing an EPS geofoam embankment. Global slope stability analyses should be performed similar to analyses performed for conventional embankments constructed with earth materials. Since the in-situ foundation material will often be saturated, cohesive soil, both undrained and drained shear strength parameters must be used in the analyses.

Similar to the settlement analyses, slope stability analyses should be performed to not only verify the stability of the proposed embankment configuration, but to also optimize the amount of EPS geofoam used (because of its high cost relative to conventional embankment materials). In many cases, settlement will control the amount of geofoam needed in the embankment cross-section, and the slope stability analyses are performed to verify stability of

the proposed embankment cross-section. However, when settlement does not control, and when practical and economical, use the slope stability analyses to help determine the most appropriate embankment configuration and placement location of the EPS geofoam.

The difficulty in performing the global slope stability analyses is how to most accurately model the shear strength of the EPS geofoam embankment. NCHRP Web Document 65 and Project No. 24-11(02) Final Report presents several methods to model the EPS geofoam embankment. However, each method has shortcomings, and there is currently no widely recommended or accepted method. Based on review of these methods and independent analyses, the methods discussed below must be used to analyze slope stability of geofoam embankments for Department projects:

1. The foundation material supporting the EPS geofoam must be analyzed by modeling the geofoam, cover materials, pavement section, traffic surcharge and any other loads as a uniform surcharge load applied at the base of the geofoam fill. This method eliminates the need to assign shear strength parameters to the EPS geofoam. For a typical embankment cross-section with side slopes, the surcharge load will vary along the base of the geofoam due to variation in cover thickness materials, extent of pavement section, traffic surcharge and other possible loads. Load distribution through the geofoam should not be used when calculating the surcharge load to be applied at the base of the geofoam.

As an example, assume a 20-foot high geofoam embankment is proposed. The embankment has 2H:1V side slopes, and a minimum of 4 feet of embankment cover (measured perpendicular to slope face) will be placed on the side slopes. Three feet of cover will also be placed over the geofoam beneath the pavement section, and a 2-foot thick pavement section is anticipated. The surcharge loads applied at the base of the geofoam fill for the slope stability analysis would consist of an approximately 600 pounds per square foot (psf) uniform surcharge beneath the side slopes of the embankment, and an approximately 1,000 psf uniform surcharge beneath the roadway/pavement portion of the embankment. The 600 psf surcharge accounts for approximately 5 feet of embankment cover material (measured vertically over side slopes) and the negligible weight from the geofoam. The 1,000 psf surcharge accounts for 3 feet of embankment cover, 2-foot thick pavement section, traffic surcharge load of 360 psf and negligible weight from the geofoam.

2. In addition to analyzing the geofoam foundation material as discussed above, the internal stability of the geofoam embankment and compound failures extending through the geofoam embankment and into the foundation material must be considered. These failure modes must be analyzed by modeling the EPS geofoam with a shear strength value consisting of a combination of friction and cohesion to simulate failure both along joints between blocks and through individual blocks. NCHRP Web Document 65 suggests using a shear strength consisting of 25% of the cohesion of a geofoam block, and 75% of the interface friction angle between geofoam blocks. The geofoam block cohesion is assumed to equal half of the

compressive resistance at 1% strain, and a block interface friction angle of 30 degrees is commonly used. Therefore, for Type 1 geofoam (i.e., EPS22), the shear strength used in the slope stability analysis to model the geofoam would consist of cohesion equal to 0.9 psi (i.e., 25% of 3.65 psf) and an internal friction angle equal to 22.5 degrees (i.e., 75% of 30 degrees).

Note that if seismic slope stability analyses are required, Method 1 discussed above cannot be used because, in a pseudo-static analysis, the seismic force must be applied at the center of gravity of the sliding mass.

13.6.3 EPS Geofoam Compressive Resistance

Since all EPS geofoam is extremely lightweight, the compressive resistance of the EPS geofoam, and not the density, will likely be the property that controls the geofoam type required for a specific project and application. In order to estimate the required compressive resistance of the EPS geofoam, the vertical load(s) proposed to be placed on it must be estimated. These vertical loads typically fall into two areas (however other load conditions that may occur must be considered). The first are service loads comprised of (but not necessarily limited to) dead loads from embankment cover, pavement and traffic live loads. The second are construction live loads from construction equipment used during placement and compaction of cover materials and pavement. Some combination of construction loads with already placed cover materials may also control. Note that construction live loads will typically control, but all combinations must be considered to determine the critical loading condition.

ASTM D6817 includes physical property requirements for seven types of EPS geofoam, and these are also shown in [Table 13.3-1](#). Compressive resistance requirements for EPS geofoam are reported at strains of 1%, 5% and 10%. As indicated in ASTM D7180, the compressive resistance at 1% strain should be used to select the appropriate EPS geofoam type for the anticipated long-term loads. The compressive resistance at 1% strain, which is also referred to as the elastic limit stress, provides acceptable short-term deflections and limits long-term creep deformation. The compressive resistance at 5% or more strain is into the plastic behavior region of the EPS geofoam where long-term creep deformation can be a problem.

Detailed discussions on the procedure for estimating vertical load applied to the EPS geofoam from dead and live load is included in NCHRP Web Document 65 and Project 24-11(02) Final Report. However, these NCHRP reports only address the service condition; they do not discuss loading conditions during construction. Both service and construction loading conditions are discussed below.

13.6.3.1 Live Load

The most severe live load applied to roadways is typically wheel loads from large trucks, and these loads can result in pressures of over 100 psi at the point of load application. These pressures are well above the compressive resistance of EPS geofoam, and therefore is the reason large trucks and other equipment are not permitted directly on the surface of EPS geofoam. Wheel loads do, however, dissipate quickly with depth due to load distribution, which is

commonly assumed to occur at a ratio of 1H:2V or flatter, depending upon the material the load is distributed through (e.g., soil, aggregate, concrete, etc.). Consider a dual tire with a contact area of 100 square inches at the surface (i.e., point of application), and a pressure distribution of 1H:2V with depth, which is commonly used for soil/embankment. The distributed contact area at a depth of 6 inches is nearly three times the contact area at the surface, and the distributed contact area at a depth of 18 inches is over eight times the contact area at the surface. The load distribution with depth quickly reduces the pressure from wheel loads. For instance, with 4 feet of cover material placed on top of EPS geof foam, the pressure from wheel loads is typically less than approximately 4 psi.

13.6.3.2 Dead Load

Dead load applied to EPS geof foam embankments is typically from the cover materials and pavement section that is placed on top of the geof foam. Unlike wheel loads, these dead loads are not distributed with depth due to the large area of the applied load. Considering that the typical unit weight of soil embankment and pavement ranges from approximately 120 to 140 pcf, each foot of these materials placed on top of EPS geof foam results in approximately 1 psi pressure to the geof foam. Because of this, only the minimum necessary cover materials and pavement should be placed on top of EPS geof foam and, the geof foam should always be placed in the upper portion of the embankment above the traditional earth embankment materials in order to limit the dead load placed on the EPS geof foam.

13.6.3.3 Sample Calculations

Sample calculations were prepared and are included in [Section 13.8](#). The calculations were done for conditions that are representative of a typical EPS geof foam embankment designed and constructed in accordance with Publication 408, Section 219, Geof foam Lightweight Fill, and this chapter. Construction loads/conditions and final loads/conditions were considered. These calculations represent specific project conditions and assumptions. Calculations must be performed and submitted that represent specific project conditions and needs if they differ from those presented in [Section 13.8](#). A discussion of the sample calculations is provided below:

1. Construction Condition - Case 1: As previously stated, due to the relatively low compressive resistance of EPS geof foam, construction equipment is not permitted directly on the surface. Therefore, the load from construction equipment at the surface of the geof foam was estimated after placement and compaction of one lift (i.e., 6 inches) of capping material over top of the EPS geof foam. The load from a triaxle truck was used in the calculation since these trucks are typically used to haul embankment material and are likely to be the most economical method of hauling material. Case 1 construction condition estimates the pressure on the surface of the EPS geof foam, and a summary of the calculation is presented below.

Assumptions:

- Twenty-one thousand four hundred-pound axle load per Trucker's Handbook, Publication 194, October 2010

- Axle has two sets of dual tires with a load of 10,700 pounds per dual set of tires
- Assume dual tire contact area of 20 inches by 5 inches, which corresponds to approximately 105 psi tire pressure
- Tire load distributed at 1H:2V through 6 inches of capping material on top of EPS geofoam
- No load factor applied since this is short term loading condition

Results for pressure on surface of EPS geofoam:

- Pressure from live load = 37.4 psi
- Pressure from dead load = 0.5 psi (from 6 inches of capping material)
- Total pressure = 38 psi

Conclusions:

- Total calculated pressure of 38 psi exceeds the compressive resistance of EPS22 at both 1% and 5% deformations (i.e., 7.3 and 16.7 psi, respectively, refer to [Table 13.3-1](#)); therefore, geofoam with a higher compressive resistance is needed at the top of the geofoam block fill.
- Total calculated pressure of 38 psi exceeds the compressive resistance of EPS39 at 1% deformation (i.e., 10.9 psi), and slightly exceeds the compressive resistance of EPS39 at 5% deformation (i.e., 35 psi).
- Since construction loads are short-term, it is acceptable to exceed compressive resistance of EPS at 1% deformation, but short-term loads should not exceed compressive resistance of EPS at 5% deformation.
- It is believed that the calculation was performed very conservatively, and since the calculated pressure of 38 psi is nearly equal to the 5% deformation strength, EPS39 or stronger is acceptable to use on the surface of the EPS geofoam block fill.

2. Construction Condition - Case 2: Same construction condition as Case 1 except the pressure was estimated 1 foot below the surface of the EPS geofoam. Additionally, the assumption was made that the tire load distributed through the EPS geofoam on a 1H:2V per NCHRP Web Document 65. A summary of this calculation is presented below.

Results for pressure 1 foot below the surface of the EPS geofoam:

- Pressure from live load = 12.2 psi
- Pressure from dead load = 0.5 psi
- Total pressure = 13 psi.

Conclusions:

- Total calculated pressure of 13 psi exceeds the compressive resistance of EPS22 at 1% deformation (i.e., 7.3 psi), but not the compressive resistance of 5% deformation (16.7 psi).

- Since construction loads are short-term, it is acceptable to exceed compressive resistance of EPS at 1% deformation, but short-term loads should not exceed compressive resistance of EPS at 5% deformation.
 - EPS22 is acceptable at a depth of 1 foot below the top of the geofoam fill (i.e., below 1-foot-thick layer of EPS39) based on the assumed/modeled construction loads.
3. Final Condition: Calculations were performed to estimate the pressure on the surface of the EPS geofoam (i.e., top of EPS39) and 1 foot below the surface of the EPS geofoam (i.e., top of EPS22) at the end of construction once the cover material, including pavement section, are in place and traffic is on the roadway. Calculations were performed for both 4 feet and 6 feet of cover over the EPS geofoam. This cover thickness includes the pavement section, embankment and capping material. A summary of these calculation is presented below.

Assumptions:

- Use same axle/dual tire load used in Construction Condition – Case 1 and 2.
- Increase live load by 20% for 4 feet of cover and 10% for 6 feet of cover (i.e., dynamic load allowance factor (IM)) per DM-4, Part B, 2015, Section 3.6.2.2.)
- Use unit weight of 135 pcf for cover material, which is relatively high to account for future addition of wearing surface.

Results with 4 feet of cover:

- Pressure from live load = 3.6 psi/2.5 psi (surface/1-foot below surface)
- Pressure from dead load = 3.8 psi (surface and 1-foot below surface)
- Total factored pressure = 7.4 psi/6.3 psi (surface/1-foot below surface)

Results with 6 feet of cover:

- Pressure from live load = 1.7 psi/1.3 psi (surface/1-foot below surface)
- Pressure from dead load = 5.6 psi (surface and 1-foot below surface)
- Total factored pressure = 7.3 psi/6.9 psi (surface/1-foot below surface)

Conclusions:

- The estimated pressures at the surface of the geofoam fill (i.e., top of EPS39) of 7.4 and 7.3 psi are well below the compressive resistance of EPS39 at 1% deformation; therefore, EPS39 can resist the long-term loads/conditions modeled in the calculations.
- The estimated pressures 1 foot below the surface of the geofoam fill (i.e., top of EPS22) of 6.3 and 6.9 psi are below the compressive resistance of EPS22 at 1% deformation; therefore, EPS22 can resist the long-term loads/conditions modeled in the calculations.
- More than 6 feet of cover, including pavement section, embankment and capping material, will overstress the EPS22 geofoam.

13.6.3.4 Selection of EPS Geof foam Type Based on Calculation Results

Project specific versions of the calculations discussed above must be used to select the EPS geof foam type. As previously discussed, the long-term pressure on the EPS geof foam cannot exceed the compressive resistance of the geof foam at 1% strain. The calculation performed for the Final Condition indicates that the maximum estimated long-term pressure on the surface of the EPS geof foam is approximately 7 psi based on the assumed thickness of cover material and pavement section. Based on ASTM D6817, EPS22 geof foam has a compressive resistance of 7.3 psi at 1% strain, and is therefore adequate for this final loading condition. However, the calculations also indicate maximum short-term pressures of 38 psi on the surface of the geof foam, and 13 psi at one foot below the surface during construction. The compressive resistance of EPS22 geof foam is 16.7 psi at 5% strain, which is considerably less than the estimated construction pressure of 38 psi. Therefore, in order to maintain short-term pressures below the compressive resistance at 5% strain, EPS39 geof foam, which has a compressive resistance of 35 psi at 5% strain, must be used at the surface. Note that the calculations performed are believed to be conservative, and therefore it is acceptable that the calculated short-term pressure slightly exceeds the 5% compressive resistance. Since the estimated pressure of 13 psi at one foot below the surface is less than the compressive resistance of EPS22 geof foam at 5% strain, EPS22 geof foam is acceptable below the one foot layer of EPS39 geof foam.

Based on these calculations, Publication 408, Section 219, Geof foam Lightweight Fill requires that a one foot thick layer of EPS39 geof foam be used at the surface of the geof foam block fill, and EPS22 geof foam be used for the remainder of the geof foam embankment. Note that EPS39 is not needed to cap the side slopes of the EPS22 geof foam fill since triaxle trucks will not have adequate space to operate in this area. If specific project loads/pressures warrant the need to use other EPS geof foam types, calculations must be prepared to justify the types that are recommended. For example, the calculation discussed above was based on between 4 feet to 6 feet of cover material. If more cover material is used, EPS22 geof foam will most likely not have sufficient compressive resistance and will not be able to be used. If such a situation were to occur, a special provision would be necessary to either require the use of a **project specific approved**, higher grade, EPS geof foam (i.e., EPS29 or greater as necessary) for the geof foam below the 1 foot EPS39 geof foam cap layer, or a concrete distribution slab constructed on top of the EPS39 geof foam. Depending upon the thickness of fill above the geof foam, the 1 foot capping layer may also require a higher-grade EPS geof foam. Whichever approach is used, calculations are required to demonstrate that both temporary and permanent loads do not exceed acceptable stress levels for temporary and permanent conditions for all layers of geof foam blocks. Note that the Department recommends the concrete distribution slab be used. Ensure that any load from this slab be accounted for when calculating the pressure on the EPS geof foam.

It is worthwhile restating that the compressive resistance of EPS geof foam increases as the density increases, and the cost of the EPS geof foam increases as the density increases. This cost increase is typically significant. Therefore, from an economics point of view, it is important to specify an EPS geof foam type that has adequate compressive resistance, but not an overly conservative (i.e., high) compressive resistance. Simply put, optimize the design to strike the proper balance between strength requirements and cost. This may include the use of a load distribution enhancement such as geocell if the cost analysis justifies this type of treatment.

However, minimum fill cover must still be maintained for thermal design considerations and needs.

13.6.4 Other External Stability Analyses

In addition to external stability analyses for settlement and slope stability, the NCHRP documents discuss external stability for bearing, sliding, overturning and uplift resistance, and seismic stability. It is anticipated that these external stability issues will not typically control the design of EPS geofoam embankments on Department projects; however, all external stability modes must be considered during design, providing calculations as necessary to verify the controlling design mode.

It is anticipated that bearing will not control if the use of geofoam is for settlement mitigation. If there is fill behind the geofoam blocks, say for example where blocks are part of a sidehill fill or part of a retaining structure, sliding and possibly overturning must be checked. If a high water table exists or the application is in a tidal region (i.e., fluctuating ground water table) then uplift may need to be addressed.

13.6.4.1 Bearing Resistance

The bearing resistance of embankment foundations is typically independently addressed by the settlement and global slope stability analyses for most situations. For example, if settlement calculations indicate excessive foundation settlement is anticipated, it is likely that an analysis would indicate that the foundation material has insufficient bearing resistance to support the proposed embankment/structure. Similarly, if the global (i.e., deep seated) slope stability safety factor of an embankment is less than required, it is also likely that an analysis would indicate that the foundation material has insufficient bearing resistance. The bearing capacity/resistance analysis method presented in the NCHRP documents is the same basic concept as the bearing resistance for spread footings on soil that is presented in DM-4, and this general approach should be used if a bearing resistance analysis is necessary (especially where treatment for settlement involves soft foundation materials).

13.6.4.2 Sliding and Overturning Resistance

NCHRP also discusses analyses to ensure that the EPS geofoam embankment has sufficient sliding and overturning resistance from unbalanced water pressure, wind and lateral earth pressure.

As discussed later, geofoam used on Department projects should generally not be placed below groundwater or the anticipated high water elevation (i.e., 500-year storm); consequently, unbalanced hydrostatic pressure should rarely occur. If unbalanced hydrostatic pressure develops it will likely be in applications when geofoam is used behind a retaining wall or abutment, and not for a typical embankment section. Therefore, similar to any retaining wall analysis, unbalanced hydrostatic pressure must be accounted for during design.

NCHRP indicates sliding or overturning of EPS geofoam embankments from wind pressure is very unlikely. The weight and shear strength of cover materials make such a failure mechanism difficult. Therefore, at this time failure modes resulting from wind loads do not have to be considered unless the embankment will be exposed to hurricane force winds, or there are other significant factors of concern. Note that this is relative to final design conditions, not construction conditions. It is the contractor's responsibility to temporarily secure EPS geofoam blocks during construction to prevent dislodging from wind or other forces. Wind loads during construction do not have to be considered as part of the design, however, contractor responsibility for wind loads, and necessary securing of geofoam blocks, must be clearly indicated in the construction provisions.

Lateral earth pressure from soil or aggregate backfill placed **behind** EPS geofoam must also be considered during design. For example, if EPS geofoam is used to backfill behind an abutment or retaining wall and embankment material (Publication 408, Section 206) is used to construct the remainder of the embankment cross-section, lateral earth pressure from the embankment material may be applied to the geofoam which in turn will transfer the lateral load to the wall or abutment. As indicated by NCHRP lateral earth pressure should be conservatively assumed to be transmitted without dissipation through the geofoam to the back of the abutment/wall. In order to avoid developing lateral earth pressure, the embankment material behind the EPS geofoam must be placed at a slope that is independently stable, such as 2H:1V (i.e., sufficiently beyond the active earth pressure wedge such that no lateral loads can be transferred to the wall).

Mechanical connectors, typically small steel plates with barbs, are available and sometimes used to improve the sliding resistance between EPS geofoam blocks, particularly from seismic loading. Based on current available information, the actual effectiveness of these mechanical connectors is questionable, and therefore, generally should not be required on Department projects. They may however be used by the contractor, at their discretion, to facilitate placement of the blocks and help prevent movement during construction. There is no known negative effect of using mechanical connectors, but they likely will increase the cost of the EPS geofoam embankment. Metal stamped mechanical connectors may not be considered in the design for stability of geofoam applications. If some type of mechanical fastening is required for final design conditions, or to facilitate use of geofoam due to loads from construction sequencing, these must be specifically designed, and their adequacy demonstrated for the specific situation and application. An alternative to mechanical connectors is the use of shear keys. As indicated in NCHRP Project No. 24-11(02) Final Report, these shear keys consist of half-height blocks of EPS geofoam that are periodically placed to interrupt the horizontal joints that are typical in most geofoam embankments.

13.6.4.3 Hydrostatic Uplift Resistance and Buoyancy Considerations

EPS geofoam is susceptible to uplift when submerged in water because of its extremely low weight/density. The density is so low that a 4-foot-thick block of EPS22 geofoam will only penetrate approximately 1 inch below the surface of water, while the remaining approximately 47 inches will stay above the surface of the water. Consequently, when geofoam is used on Department projects, the bottom of the geofoam should be placed above the static groundwater

level and above the anticipated high water level (i.e., 500-year storm elevation) if used in the vicinity of a stream, river, etc. However, if it is necessary to place any part of the geofoam embankment below groundwater or the anticipated high water level, an analysis must be performed to ensure all other options have been considered, and that sufficient cover material will be placed over the EPS geofoam embankment to prevent uplift. Also, anchorage of the blocks may be necessary to facilitate construction. Traffic loads and any other temporary or short-term loads should not be included when calculating the uplift resisting force. Use a minimum safety factor of 1.2 if the hydrostatic uplift force is a short-term condition; a safety factor of 1.5 is required if the uplift force is a long-term condition, such as when any part of the EPS geofoam is submerged beneath the static/long-term groundwater level or on cyclical basis in tidal areas, or similar regularly fluctuating groundwater level.

If any part of the EPS geofoam embankment may become submerged during the life of the embankment, buoyancy calculations must be performed. These calculations must consider final, temporary (i.e., high water level during flood event, tidal conditions, seasonal ground water fluctuations) and short-term (i.e., during construction) conditions, and both normal and high water levels must be used in the analyses. Soil cover must be used to overcome buoyancy forces for final and temporary conditions; anchoring is not permitted to resist final and temporary buoyancy forces. Anchoring is permitted to resist buoyancy forces that develop during short-term conditions. If analysis indicates tie down anchors or other mechanical means are necessary to prevent uplift, then the use of EPS geofoam should be reconsidered due to the additional cost, effectiveness, and long-term viability of these mechanical means. Use tie downs or other mechanical means only after discussion with and approval from the District Geotechnical Engineer (DGE).

13.6.4.4 Seismic Stability

Seismic stability analyses should also be considered when using EPS geofoam as an embankment material. In general, follow the same basic Department seismic slope stability procedures to analyze EPS geofoam embankments that are used to analyze conventional embankment/backfill materials. The NCHRP reports also provide guidance on performing these analyses. Seismic global slope stability should be considered for EPS geofoam embankments with both sloping and vertical (i.e., wall) sides. When EPS geofoam is used as a retaining wall backfill, seismic overturning and internal stability should also be considered. Based on information presented in NCHRP, and since the peak horizontal ground acceleration in most of parts of Pennsylvania are low, seismic conditions will not likely control the design of the EPS geofoam embankments.

13.6.5 Cover Requirements

EPS geofoam must be covered with a minimum of 4 feet of cover material, which includes 6 inches of capping material, soil embankment material, and the pavement section as shown in [Figure 13.7-1](#). However, in order to not overstress the EPS22 geofoam, the maximum combined thickness of the cover material (i.e., capping material, soil embankment material and pavement section) that is placed over the EPS geofoam should not exceed 6 feet. If it is not possible to reduce the total, maximum, combined thickness to 6 feet, then EPS geofoam with a

compressive resistance higher than the compressive resistance of EPS22 must be used. Costs for the denser geofoam should be considered and balanced against needs of thicker cover materials, and/or design of surcharge loads to limit resulting pressures on geofoam.

The cover material is needed because EPS geofoam acts as an insulator by preventing heat from the ground reaching the pavement section. Premature freezing of the roadway surface can occur if an inadequate thickness of cover material is used between the EPS geofoam and pavement section. Additionally, the cover material distributes wheel loads applied to the pavement which reduces the stress on the EPS geofoam. Four feet of embankment cover is also needed over the EPS geofoam on embankment side slopes and over areas that pavement will not be constructed as shown in [Figure 13.7-3](#).

13.6.6 Hydrocarbon Resistant Geomembrane

Since EPS geofoam is petroleum based material, it will degrade when in contact with many petroleum products, organic solvents and some vegetable based oils, including the vapors from these products. Liquids and their vapors that will degrade EPS geofoam include but are not limited to gasoline, diesel, kerosene, mineral spirits, acetone and benzene. Therefore, during design of an EPS geofoam embankment, the design must accommodate the potential of a spill of these types of products on the roadway, and the consequences of damage to the EPS geofoam. A hydrocarbon resistant geomembrane must be used to protect the EPS geofoam from spills of these harmful liquids on the roadway. While the cost of a geomembrane protection envelope is not insignificant, the cost of repair and service interruption as the result of spill, are much greater.

As indicated in Publication 408, Section 219, Geofoam Lightweight Fill, the hydrocarbon resistant geomembrane must be listed in Bulletin 15 and manufactured from a tri-polymer consisting of polyvinyl chloride, ethylene interpolymer alloy and polyurethane, or a comparable polymer combination. When a geomembrane is used, it will typically encapsulate the entire EPS geofoam mass in order to protect the geofoam from damage from both liquids and vapors as discussed in [Section 13.7.9](#). An alternate system, such as a geomembrane over just the top surface of the geofoam can be considered. However, the alternate system must provide the level of protection similar to the protection provided by complete encapsulation, and must accommodate the collection and handling of any liquids and vapors. This may include the need for special drainage, venting or other methods of control and/or protection. Details must be developed to seal around any drainage pipes, inlet boxes or other control devices and/or obstructions. As vapors from spills can be just as damaging as direct liquid contact, full encapsulation with a hydrocarbon resistant membrane is the preferred method of protection.

The NCHRP publications discuss the possibility of using a lightly reinforced, 4-inch thick concrete slab to protect the EPS geofoam from spills of harmful liquids. Inevitable cracking of the concrete slab makes a geomembrane a much more effective option to protect the EPS geofoam from spills of damaging solvents. Additionally, a geomembrane provides the ability for total encapsulation, which the concrete slab cannot. Therefore, on Department projects, the use of a concrete slab to protect the EPS geofoam from harmful spills should not be used. However, a concrete slab can be used if needed for other design purposes, such as to help distribute wheel loads.

13.6.7 Layout, Transitioning and Side Slopes

During design the geotechnical designer is responsible for determining the limits of the EPS geofoam within proposed embankments based on the results of the external stability analyses (e.g., settlement, global slope stability, etc.). The geotechnical designer must coordinate with roadway/structure designer(s) to show these limits on the contract drawings (e.g., plans, cross-sections, details, etc.). The geotechnical designer should not specifically detail the size and orientation of each block. Block size can vary between manufacturers, and Publication 408, Section 219, Geofoam Lightweight Fill indicates minimum block size and required block placement orientation. Furthermore, Publication 408, Section 219.3(c) requires the contractor to submit a shop drawing prior to placing blocks that shows the location and orientation of each block. Numerous requirements in the specification must be met with respect to orientation of blocks and location of joints between blocks. Only the limits and width, height and slope requirements should be indicated for the geofoam block fill, but not individual block size or orientation.

Due to the dissimilar properties of EPS geofoam and typical embankment materials, and to avoid differential settlement, abrupt changes between these materials should be avoided. In the longitudinal direction of the roadway, gradually transition from embankment to EPS geofoam. The NCHRP report indicates that the transition between the EPS geofoam and embankment should be designed so that differential settlement does not exceed 1:200 (vertical to horizontal). In the transverse direction, particularly when used behind retaining walls, it is recommended to use EPS geofoam beneath the entire paved roadway width to avoid a crack in the pavement from differential settlement between EPS geofoam and embankment material. If not used full width, then gradual tapering must be required. All transitions must be carefully designed to prevent abrupt changes in base stiffness and load response.

When using EPS geofoam to construct typical “trapezoidal” shaped embankments, design the side slopes of the embankments no steeper than 2H:1V. The embankment cover material will not typically have adequate strength to provide sufficient veneer stability for slopes steeper than 2H:1V. Uniformly step the EPS geofoam on the side slopes, similar to typical soil benching details, to help improve stability of the cover material. Do not trim the EPS geofoam blocks to match/parallel the final slope of the embankment. If embankment slopes steeper than 2H:1V are proposed, special details must be developed and analyses must be provided to verify that the embankment cover on the side slopes will be stable. EPS geofoam can be designed with a vertical face and be internally stable, but structural support/facing is required. If it is known that native materials are inadequate for a 2H:1V slope, then design accordingly by either flattening the slopes or providing treatment measures and details.

13.6.8 Drainage Pipes, Utilities and other Typical Roadway Features

Roadway embankments typically contain drainage pipes and inlet boxes, and often also have underground utilities, utility poles, light poles, signs, guide rail posts and other appurtenances. These can be incorporated into an EPS geofoam embankment, but careful consideration should be given during design. When possible, locate these appurtenances outside

the limits of the EPS geofoam or in the embankment cover material above the EPS geofoam. Drainage pipes, utilities and other features can be located within the EPS geofoam; however, this will require special fabrication of blocks at the factory or field cutting to fit blocks around obstructions, which can be costly. Additionally, geomembrane protection for the EPS geofoam is more difficult and costly to install around obstructions. A concrete slab may be required to support light poles, signs and other heavier features, but any load from this slab must be accounted for when calculating the pressure on the EPS geofoam.

13.7 CONSTRUCTION REQUIREMENTS AND CONSIDERATIONS

Construction requirements for the use of EPS geofoam to construct roadway embankments are provided in the Publication 408, Section 219.3. This section provides background information for some of these requirements. Additionally, a typical profile view is shown in [Figure 13.7-1](#), a typical section is shown in [Figure 13.7-2](#), a side slope detail is provided in [Figure 13.7-3](#), and a detail of the hydrocarbon resistant membrane is shown in [Figure 13.7-4](#).

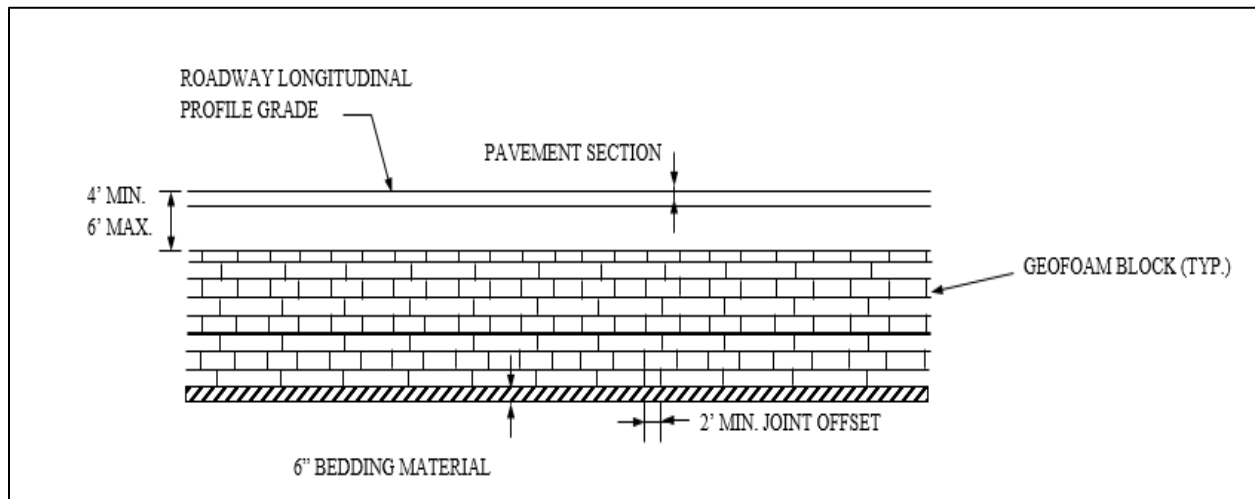


Figure 13.7-1 – Typical Longitudinal Profile View

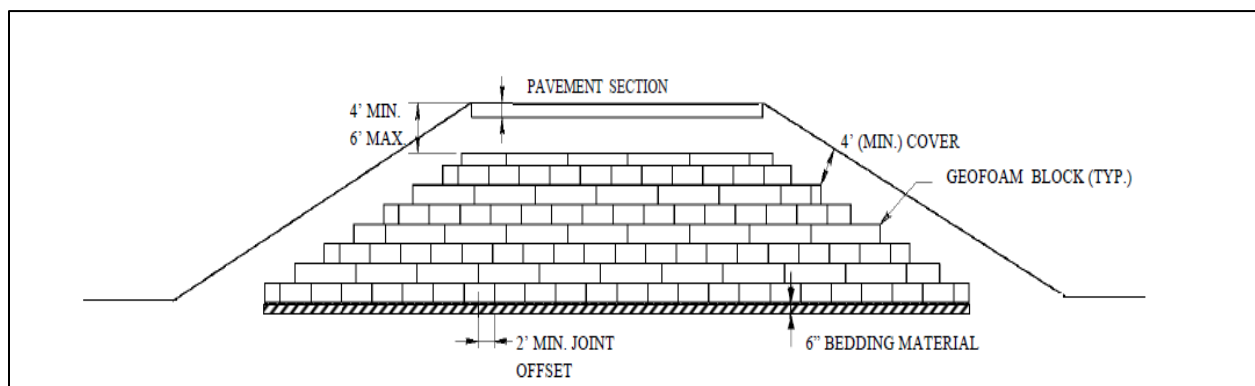


Figure 13.7-2 – Typical Embankment Section

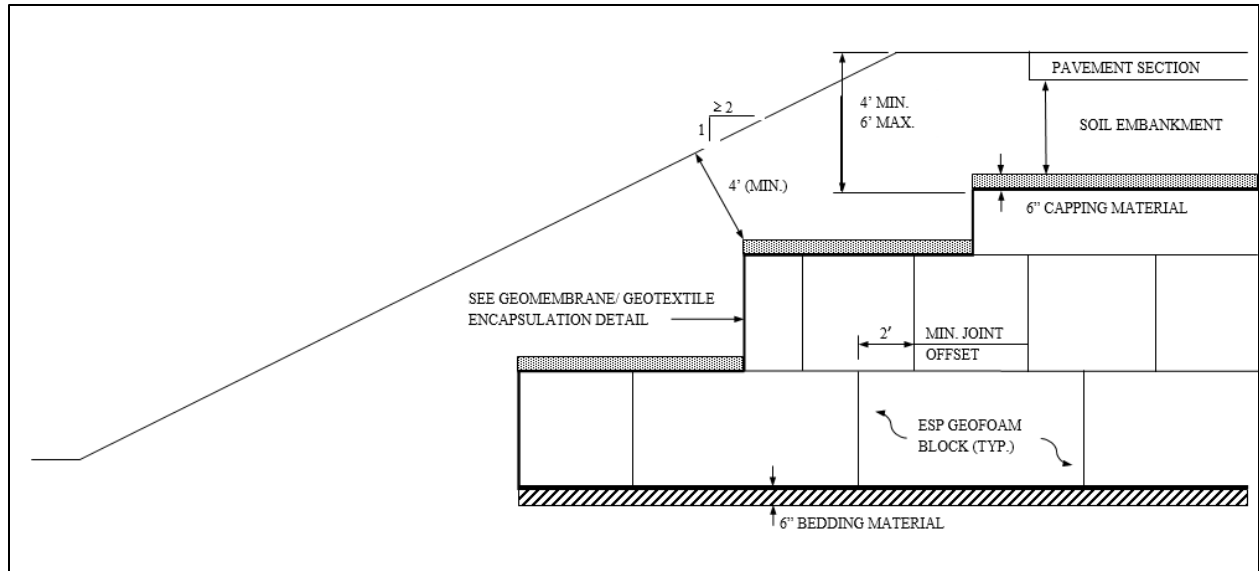


Figure 13.7-3 – Side Slope Detail

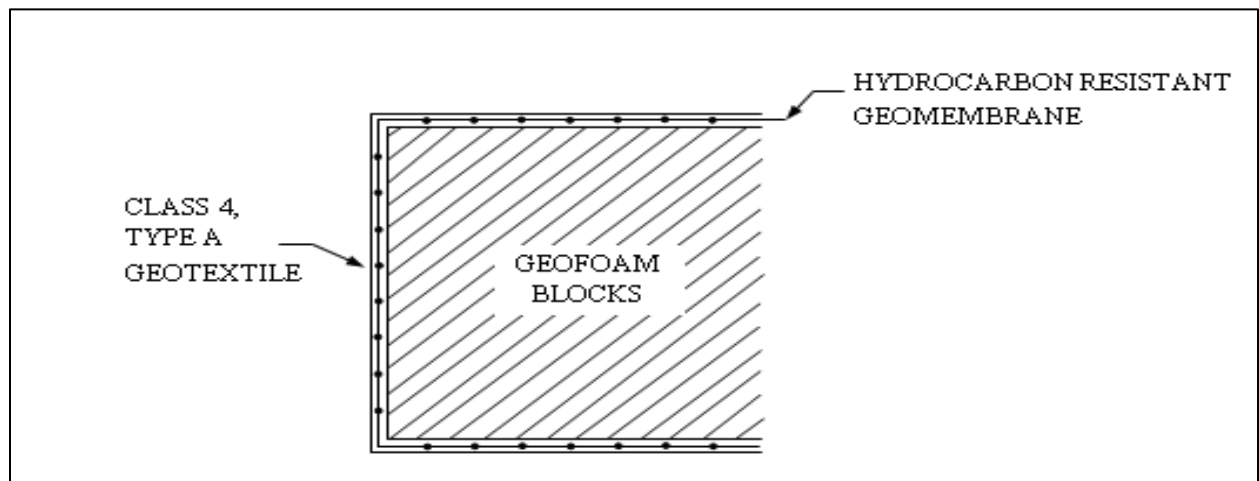


Figure 13.7-4 – Geomembrane Encapsulation Detail

13.7.1 Material Requirements

EPS geofoam used on Department projects must be supplied by a manufacturer that is listed in Bulletin 15, and must meet the requirements of ASTM D6817 and Table A as indicated in Publication 408, Section 219.2(a). As previously discussed, calculations indicate that geofoam types EPS22 and EPS39 will be required for most Department projects. EPS22 will typically be used to construct the majority of the geofoam embankment with the exception of the top one foot where EPS39 geofoam must be used. Other EPS types (i.e., densities) can be used if necessary and justified, and if calculations demonstrate its adequacy. If other EPS types are necessary, a job specific specification must be developed most likely consisting of revisions to Publication 408, Section 219. Additional material requirements are discussed below.

13.7.1.1 Dimensions, Squareness, and Planarity

Publication 408, Section 219.2(a), Table A requires blocks to have minimum dimensions of 4 feet wide by 8 feet long. The EPS22 blocks must be at least 2 feet high, and the EPS39 blocks used at the top of the EPS geofoam fill must be at least one foot high. EPS geofoam blocks are manufactured in a variety of sizes, but these minimum required dimensions are commonly used for roadway projects, and most manufacturers are capable of supplying blocks with these dimensions. The minimum dimensions are specified to help limit the number of joints and meet joint spacing/location requirements, which are intended to help construct an internally stable embankment. Blocks with dimensions smaller than these can be supplied where necessary for project specific fill geometry (i.e., such as to complete a row or layer of blocks or for block placement around obstructions).

EPS geofoam blocks must be manufactured with perpendicular and planar faces, except as needed for project specific fill geometries. The corner or edge of all adjacent faces must be perpendicular within the tolerance indicated in Publication 408, Section 219.2(a). Perpendicular faces are required so adjacent blocks fit tightly with no or little open space between blocks. The faces of the blocks must also be planar (i.e., flat and not warped) within the tolerance indicated by the specification. This is required so loads are uniformly distributed to the entire block face/surface and not to just high spots on the block. This is also required so adjacent blocks fit tightly with no or little open space between blocks. Flat top surfaces also prevent ponding water from precipitation that may occur during placement of the blocks.

13.7.1.2 Regrind

EPS geofoam blocks must be manufactured with virgin polystyrene and not regrind material (i.e., recycled geofoam). It is currently unclear what the effects of the use of regrind are on the properties of EPS geofoam blocks, or what percent regrind is appropriate to use. Should adequate, reliable information become available concerning the impact of regrind on the performance of EPS geofoam blocks, the use of regrind in the manufacture of EPS geofoam for Department projects will be re-evaluated.

13.7.1.3 Flammability

EPS geofoam blocks must be manufactured with a flame retardant to meet the flammability requirements (i.e., minimum oxygen limit) indicated in Table A of Publication 408, Section 219.2(a). Flame retardant is used specifically to prevent fires during construction because once the EPS geofoam is covered with embankment the potential for ignition of the geofoam is very low. While the use of flame retardant does increase the cost of manufactured geofoam blocks, it is worthwhile insurance relative to the cost associated with a fire in a geofoam fill. Note that the flame retardant prevents the EPS geofoam from burning but it does not prevent it from melting. Additionally, the flame retardant does not prevent fires from outgassing. Outgassing is discussed below in [Section 13.7.2](#), Seasoning.

13.7.1.4 Insecticide

An insecticide must also be included in the geofoam blocks to prevent attack from termites and other insects. No documented cases of damage from insect attack on EPS geofoam fills was found, but insect attack on geofoam used in the building construction industry is known. Insecticide is commonly used and is not expected to significantly increase the cost of manufacturing the EPS geofoam blocks. Note that if the EPS geofoam mass is fully encapsulated with a geomembrane, insecticide is not needed since the geomembrane would provide an effective barrier from insect attack.

13.7.2 Seasoning

As discussed earlier in [Section 13.1](#) a gas, called a blowing agent, is used to manufacture EPS geofoam. A variety of gases can be used, including pentane and butane. Immediately after the geofoam is manufactured (i.e., released from the mold), the blowing agent begins to “escape” from the blocks. This process is called outgassing, and seasoning is the term used to describe the period of time required for the majority of the blowing agent to escape the EPS geofoam. If EPS geofoam blocks are placed in a large, tightly placed mass, such as a roadway embankment, before the blocks are seasoned, the gas can become trapped in the joints between blocks and ignite, melting and damaging the geofoam.

The seasoning time required for EPS geofoam is not definitive. A period of 72 hours is typically recommended, and is the minimum seasoning period required by Publication 408, Section 219.2(a). The seasoning time is dependent upon several factors, including air temperature and air circulation. EPS geofoam blocks season faster at higher temperatures. Publication 408, Section 219.2(a) requires a minimum temperature of 68 degrees Fahrenheit in the seasoning room/facility. Additionally, the blocks must have adequate space between all faces of the block to allow the gas to escape, the room must have adequate ventilation to promote circulation of air around each block, and the seasoning room must protect the blocks from moisture and ultraviolet (UV) radiation.

13.7.3 Delivery and Storage on Site

EPS geofoam blocks are typically delivered to the site in a box trailer or on a flatbed trailer. Blocks must be protected from damage during delivery and offloading at the project site. Each block delivered to the project site must be labeled. The label must include:

- the manufacturer’s name
- ASTM EPS designation per Publication 408, Section 219.2(a), Table A
- date the block was molded
- the weight (in pounds) and density (in pounds per cubic foot) of each block as measured after the minimum required seasoning period.

The manufacturer’s name is required because on some projects, particularly large ones, more than one manufacturer may be used by the contractor to supply EPS blocks. The EPS designation is required because typically two types of EPS geofoam will be used on a project.

The date is required for quality control tracking purposes, and the weight and density are required so that blocks can periodically be weighed at the project site for QA/QC purposes. It is the contractor's responsibility to provide a scale for the Engineer's use at the project site that is appropriate for weighing EPS geofoam blocks.

If geofoam blocks are not immediately placed to construct the proposed geofoam embankment when delivered to a project, they can be temporarily stored on site. The designated storage area must be secure and located away from any heat source or construction activity that produces heat, flame, sparks, etc. Personal tobacco smoking is not permitted in the storage area. Even though the blocks contain a fire retardant, adequate measures must be in place to protect the blocks from flames and heat. EPS geofoam blocks must also be protected from contact with and exposure to vapors from organic solvents, including but not limited to acetone, benzene and mineral spirits, and petroleum based solvents, including but not limited to gasoline, kerosene and diesel fuel.

The blocks must be stored off the ground, and no part of the block can be stored in standing water. The blocks must be protected from discoloration and dusting caused by excess exposure to sunlight, but a cover cannot be used directly over the blocks, because a cover may trap heat and cause excessive temperatures that could damage the blocks. Equipment and vehicles cannot traverse the blocks, and foot traffic must also be kept to a minimum. If blocks are exposed to wind, it is the contractor's responsibility to secure the blocks with sandbags or other similar "soft" weights that do not damage the blocks.

13.7.4 Contractor Submittal

As discussed throughout [Section 13.6](#) of this publication, during design the engineer/designer must indicate the limits of the EPS geofoam block, but not the actual layout of individual blocks. It is the contractor's responsibility to prepare a submittal that details individual block placement that follows the requirements of Publication 408, Section 219.3(c). Some of these requirements are discussed below:

1. Unless specifically indicated otherwise on the contract drawings, Type 1 geofoam blocks (EPS22) will be used for the main body of the fill, and a minimum thickness of 12 inches of Type 2 geofoam blocks (EPS39) will be placed on top of the Type 1 geofoam blocks. Type 2 geofoam blocks are not required to be used on side slopes of the block fill.
2. Use the maximum amount of full size blocks that the placement geometry allows. Additionally, the blocks must be placed with their minimum dimension (i.e., thickness) oriented vertically because this orientation will produce the most internally stable geofoam embankments. Only in cases where a limited number of blocks are necessary to achieve geometric requirements should blocks be oriented other than with the minimum dimension vertical.
3. A minimum of two layers of blocks must be used at all locations unless specifically indicated otherwise on the contract drawings. This can consist of one

layer of Type 1 geofoam capped with a layer of Type 2 geofoam, or two layers of Type 1 geofoam on side slopes or under other unpaved areas. The use of only one block beneath roadways has shown signs of sliding instability and premature pavement distress. This instability was most likely where minimal cover was placed over the EPS geofoam. Due to the “thicker” cover requirements indicated in Publication 408, Section 219, sliding instability most likely will not be a problem even if one layer of blocks is used. Regardless however, to adhere to best practices, a single layer of blocks should be avoided where possible.

4. The plane on which a given layer of blocks is placed must be parallel to the longitudinal axis/profile of the roadway alignment. Essentially, the subgrade surface on which the first layer of blocks is placed must be parallel to the longitudinal profile of the roadway. This is required to avoid or reduce the need to trim EPS blocks at or near the top of the block fill to meet required tolerances. Excessive trimming of blocks will not only be time consuming and costly, but will most likely result in an overall lower quality top of block surface.
5. The longitudinal axes of the uppermost layer of blocks must be perpendicular to the longitudinal axis of the road alignment. It has been found that the blocks and pavement section perform better when the top layer of blocks are oriented in this direction.
6. Within a given layer of blocks, the longitudinal axes of all blocks must be parallel to each other, and vertical joints between adjacent longitudinal rows of blocks must be offset a minimum of 2 feet. Additionally, the longitudinal axes of blocks for layers above and/or below a given layer must be perpendicular to the longitudinal axes of blocks within that given layer. These requirements for longitudinal axis orientation and joint location are to prevent continuous joints and provide an interlocking system of blocks that is internally stable.
7. The blocks must be covered with a 6-inch compacted layer of capping material to protect the geotextile and geomembrane from damage. The total thickness of cover over top of the block fill, which includes capping material, embankment material, and pavement section, must be a minimum of 4 feet but not exceed 6 feet.

13.7.5 Damage

Care must be taken during loading, shipping, unloading, temporary storage and placement of EPS geofoam to avoid damaging the blocks. While some minor damage to the blocks may occur during normal handling procedures, excessive damage will reduce the bearing area of the blocks, which results in higher stresses on the blocks. These higher stresses may exceed the design stress of the EPS geofoam and cause unacceptable deformation. Excessive damage also may prevent the blocks from fitting tightly, which could reduce the internal stability of the EPS geofoam embankment. Excessive or unacceptable damage requires the block to be

replaced at the contractor's expense, but "undamaged" portions of these blocks can be used on the project where partial blocks are needed.

Damage includes but is not limited to dents, gouges, divots and missing corners/pieces. Publication 408, Section 219.3(d)1 defines unacceptable damage as any of the following:

1. Volumetric damage of more than 0.5% of the volume of the single block. The minimum EPS22 block size required by the specification has a total volume of 64 cubic feet; therefore, the sum of the volumetric damage to a single block cannot exceed 0.32 cubic feet for this size block.
2. Surface damage of more than 5% of the load bearing area of the single block. Using the dimensions of the minimum required block size, the load bearing area is typically 32 square feet. Therefore, no more than 1.6 square feet of damage to this load bearing surface is acceptable.
3. In addition to Item 2 above, surface damage of more than 5% of the total block surface area is unacceptable. This includes all surfaces of the block, not just load bearing surfaces.
4. Continuous damage of more than 20 % of the length of any side of a single block is also unacceptable.

Damaged areas on horizontal bearing surfaces that are acceptable per the above criteria shall be filled with dry, fine sand meeting the requirements of Publication 408, Section 219.2(d). Do not fill open vertical surfaces with sand.

13.7.6 Cutting

EPS geofoam blocks will need to be cut in order meet project specific fill geometries and to fit around obstructions. These "specialty" blocks can either be fabricated in the factory or on the project, the latter being more typical. Typically, these blocks are not molded to the required size/shape but instead they are cut from full size blocks. Numerous tools have been used to cut blocks, including a hot wire, chain saws, hand saws and other mechanical cutting and shearing devices. For Department projects, EPS geofoam blocks may only be cut in the factory or field with a hot wire.

This hot wire is made of nickel chromium (NiCr) that is connected to an electricity source. When electricity passes through the wire, it gets hot and cuts (i.e., melts) the EPS geofoam. The hot wire method of cutting allows for accurate and clean/smooth cut block faces. Chain and hand saws and other cutting methods usually do not yield cut faces as clean/smooth as the hot wire and are therefore not permitted for use on Department projects.

Geofoam blocks that are cut must be to the maximum dimensions possible, cutting full length and width (where practical), and to the necessary thickness, maintaining a smooth and level surface. Field and factory cuts must be smooth and flat on all surfaces, unless curved surfaces

are needed to fit tightly. Blocks must be cut to within 0.5 inch of the required or specified dimensions.

13.7.7 Subgrade Preparation

One of the most important steps of successfully constructing an EPS geofoam embankment is preparation of the subgrade. In general, the subgrade must be prepared in accordance with Publication 408, Section 206.3(a). It must be relatively smooth and free of any localized hard spots or protrusions that could damage the geomembrane or geofoam blocks, and it must be free of standing water. As previously discussed, the subgrade must be prepared smooth and parallel with the longitudinal profile grade of the roadway so that the horizontal surfaces of the blocks are parallel with the longitudinal profile grade. The subgrade should be level in a direction transverse to the roadway. Any cross slope for the final roadway must be developed in the cover materials above the geofoam blocks.

Once the subgrade is prepared, a 6-inch loose leveling/bedding course is placed. The bedding material can consist of AASHTO No. 10 Coarse Aggregate, Type A Cement Concrete Sand, or Type C Mortar Sand. The loose leveling course permits fine grading for the first layer of blocks, and allows the blocks to be “seated”. It is critical that the first layer of blocks be stable and level so that it is easier to maintain alignment and tolerances of remaining layers of block. Once the leveling course is prepared, geotextile and the hydrocarbon resistant geomembrane are placed.

13.7.8 Placement

The EPS geofoam blocks should be placed in accordance with the approved placement plan prepared by the contractor, and as directed in the field by the Representative. The main body of the fill will typically consist of Type 1 geofoam (i.e., EPS22) unless otherwise required by design. These blocks will extend from the bottom of the geofoam fill to approximately one foot below the top surface of the geofoam fill. The last/top 12 inches (minimum) of the geofoam block fill will consist of a cap block which will be addressed later in this section. Where possible, the factory skin (i.e., molded, uncut surface) of the blocks should be placed as the outer layer to help minimize the amount of water absorbed by the blocks.

Blocks must be placed by hand. Wheeled, tracked or other equipment is not permitted to be operated directly on the surface of geofoam blocks as this may result in overstressing the blocks and cause damage. Equipment, such as conveyors, lifts and cranes may be used to transport or lift blocks, as long as its use does not involve traversing the blocks with the equipment or results in damage to the geofoam blocks. Use only full-size blocks (i.e., blocks of minimum dimensions indicated in Publication 408, Section 219.2(a), Table A) except where partial or cut blocks are needed to meet project specific fill requirements. If geofoam blocks are warped/crowned but within the acceptable tolerance indicated in the specification, place these blocks with the crown upward to prevent the ponding of water.

Assure full contact between blocks so that stresses are carried by the full block bearing surface and not concentrated on portions of the block. Do not leave standing water, accumulated

snow or ice, or debris of any kind on previously placed EPS blocks prior to placement of subsequent blocks. De-icing salts, such as sodium chloride or other products, can damage/degrade EPS geofoam and are not permitted to be used.

Since geofoam blocks are extremely lightweight, they can be displaced by wind. During placement of geofoam blocks, temporarily secure them with sandbags or other similar “soft” weights that do not dent or otherwise damage blocks until the soil cover is placed. Mechanical connectors (i.e., barbed, metal plates) may be used to help prevent the movement of blocks from wind, however, the effectiveness of the connectors will be dependent upon the severity of wind conditions. The contractor may elect to use mechanical connectors to help secure the geofoam blocks, but they should not be required by the designer unless specifically needed for stability of the final embankment condition or if necessary to address some other temporary or long-term need. If used, the cost of mechanical connectors must be incidental to the geofoam blocks.

The upper 12 inches (minimum) of the geofoam fill beneath the roadway, shoulders and median will typically consist of a cap block of Type 2 geofoam (i.e., EPS39). The denser and stronger EPS 39 geofoam is required to address anticipated construction loads as previously discussed in [Section 13.6.3.4](#). EPS39 geofoam is not needed on the side slopes of the embankment since construction loads are not expected to be as high. The top layer of blocks should be placed in the same manner as the lower layers of block.

The top surface of the geofoam fill must be constructed relatively smooth so that the hydrocarbon resistant geomembrane has an even bearing surface to prevent stress concentrations and possible damage. Publication 408, Section 219.3(e) requires the top surface of the blocks to be constructed to within a tolerance of $\pm 1/2$ inch over a 10-foot interval of the design longitudinal profile of the blocks and design transverse slope of the blocks. Additionally, the finished surface of the blocks beneath pavement sections must be constructed to within ± 0.1 foot of the top of block design grade, and the finished surface of the blocks on side slopes must be constructed to within ± 0.2 foot of the top of block design grade. Any adjustments for grade can be made by trimming either Type 1 or Type 2 geofoam blocks, but the Type 2 geofoam blocks must have a minimum thickness of 12 inches.

13.7.9 Hydrocarbon Resistant Geomembrane

As previously discussed, unless an adequate alternate method of protection can be provided, the EPS geofoam fill must be completely encapsulated in a hydrocarbon resistant geomembrane. The geomembrane must be one continuous layer connected by field welding the seams with a hot wedge welder. Hot air or solvents cannot be used because they are more likely to damage the geofoam compared to using a hot wedge welder. Particular care must be taken when placing the geomembrane on the stepped side slopes of the geofoam. The membrane must be sufficiently loose so embankment placement and compaction does not stretch or otherwise damage the geomembrane. A Class 4, Type A geotextile must be wrapped around the outside of the entire geomembrane to aid in protection from damage. Any portions of the geomembrane that are damaged during construction must be repaired by welding on geomembrane patches in accordance with Publication 408, Section 219.3(d)2. A detail of the geomembrane is shown in [Figure 13.7-4](#).

13.7.10 Capping and Embankment Material

Once the blocks, geomembrane and geotextile are placed, they must be covered with a 6-inch minimum compacted lift of capping material. The capping material must be relatively fine material to minimize the risk of damaging the geomembrane during placement and compaction of the material. The capping material can consist of either:

- Material meeting the requirements of Publication 408, Section 206.2(a)1.a, Soil, except 100% of the material must pass the $\frac{3}{4}$ inch sieve.
- AASHTO No. 10 Coarse Aggregate, Type A Cement Concrete Sand or Type C Mortar Sand. Note that these are the bedding materials used for the leveling course discussed in [Section 13.7.7](#), Subgrade Preparation.

The capping material must be placed on top of the geotextile, including on the horizontal surfaces of the side slopes, prior to placement of embankment material. The capping material must be placed within two weeks after placement of the geotextile to help prevent degradation of the geotextile. The capping material should be placed and compacted in accordance with Publication 408, Section 206.3(b), except the following provisions are required to help prevent damage to the geofoam blocks and geomembrane:

1. Heavy equipment cannot be operated directly on top of the blocks. Pneumatic tired equipment must be used to spread the capping material, but sharp, sudden turns are not permitted. Tracked equipment cannot be used to spread the capping material.
2. Capping material must never be end dumped just off the blocks.
3. Capping material must be bladed to an 8-inch thick loose lift across the blocks with pneumatic tired equipment.
4. Capping material cannot be end dumped directly on the blocks and cannot be stockpiled on the block fill.
5. Capping material must be compacted using a smooth drum in static (non-vibratory) mode.
6. A single triaxle truck load of capping material can be end dumped onto previously placed and compacted capping material. The dumped material must be spread as soon as practically possible.

Once the capping material is placed and compacted, embankment material meeting the requirements of Publication 408, Section 206.2(a)1.a, Soil, can be placed to finished grade. The soil should be placed and compacted in general accordance with Publication 408, Section 206.3. Once the 6-inch compacted layer of capping material is in place, conventional earth hauling equipment (i.e., triaxial trucks or smaller) may drive over the embankment, and tracked

equipment may be used to spread the embankment material. However, similar to capping material, embankment material cannot be stockpiled or end-dumped into piles on the geofoam fill at any time because this could overstress or damage the EPS geofoam. A single triaxle load of embankment can be end-dumped on previously placed and compacted capping and embankment material, but it must be spread as quickly as practically possible.

On the side slopes of the geofoam fill, place and compact soil cover starting at the bottom of the slope in such a manner as to prevent damage to the geofoam, geomembrane and geotextile. As previously discussed, capping material must be placed over the horizontal surfaces of the side slopes prior to placement of embankment material. Embankment material is permitted to be placed directly against the vertical surfaces on the side slopes.

13.8 CALCULATIONS FOR THE DETERMINATION OF REQUIRED EPS GEOFOAM COMPRESSIVE RESISTANCE AND DENSITY

This section presents calculations for the determination of required EPS geofoam compressive resistance and density. The objective is to determine the required compressive resistance of geofoam blocks to support both Construction Loads/Conditions and Final Loads/Conditions.

13.8.1 Construction Loads/Conditions

For Construction Loads/Conditions, the following assumptions will be made:

- Triaxle trucks will be used to deliver embankment to be placed over geofoam blocks.
- Triaxle trucks will be loaded to maximum permitted weight in accordance with the 2015 Edition of Publication 194, Trucker's Handbook, Legal Wheel, Axle, and Gross Vehicle Weights.
 - Maximum weight per inch of tire is 800 pounds on any one wheel.
 - Steering axle weight cannot exceed 20,000 pounds.
 - Axles spaced less than 6 feet apart have maximum weight limit of 18,000 pounds.
 - Three- and four-axle trucks are permitted to have up to 21,400 pounds on each tandem axle.

Based on above: Use maximum permitted axle load (L_{max}) = 21,400 pounds.

Note: Since construction loading is temporary/short-term, do not apply load factor for impact/dynamic loading.

- For wheel/tire contact area, use similar approach to "Concrete Pipe Handbook," American Concrete Pipe Association, 1981.
 - Typical tire width for triaxle is 9 inches. For dual tires assume 20-inch width.
 - Typical tire pressure is 80 to 100 psi.

Note: Based on discussions with some quarry owners, triaxial tire pressures typically run between 100 and 110 psi. Thus, to be conservative use a tire pressure (P_t) of 105 psi.

Therefore, with a maximum permitted axle load of 21,400 pounds, the load on a dual set of tires is determined as follows:

$$\frac{L_{max}}{\text{No. sets of dual tires}} = \frac{21,400 \text{ lbs}}{2 \text{ sets of dual tires}} = \frac{10,700 \text{ lbs}}{\text{set of dual tires}}$$

To determine dual tire contact area (A_t), use the following equation:

$$A_t = \frac{L_{max}/\text{set of dual tires}}{P_t} = \frac{10,700 \text{ lbs}}{105} = 102 \text{ in}^2$$

As previously indicated, assume a single tire width of 9 inches and a dual tire width (W) of 20 inches (refer to [Figure 13.8.1-1](#)). Therefore, tire contact length is determined as follows:

$$L = \frac{A_t}{W} = \frac{102 \text{ in}^2}{20 \text{ in}} = 5 \text{ in}$$

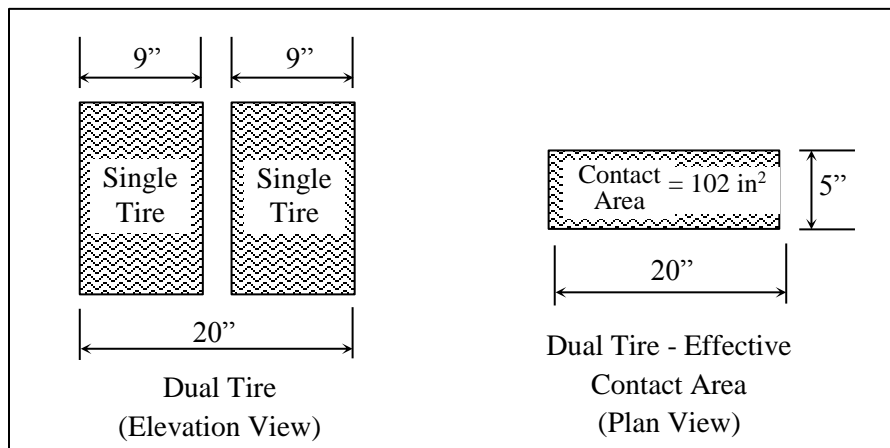


Figure 13.8.1-1 – Dual Tire Elevation View and Plan View

Proceed with estimating the pressure on the top layer of geofoam from dual tires of a triaxle dump truck by considering the following:

- Triaxle dump trucks and other “heavy” equipment cannot operate directly on top of the geofoam because the pressure would exceed the compressive resistance of the geofoam, resulting in damage to the EPS blocks.
- In accordance with Publication 408, Section 219.3(g), one layer of capping material must be placed (by blading the material) and compacted to 6 inches prior to operating triaxle trucks and other “heavy” equipment on the geofoam embankment (refer to [Figure 13.8.1-2](#)).

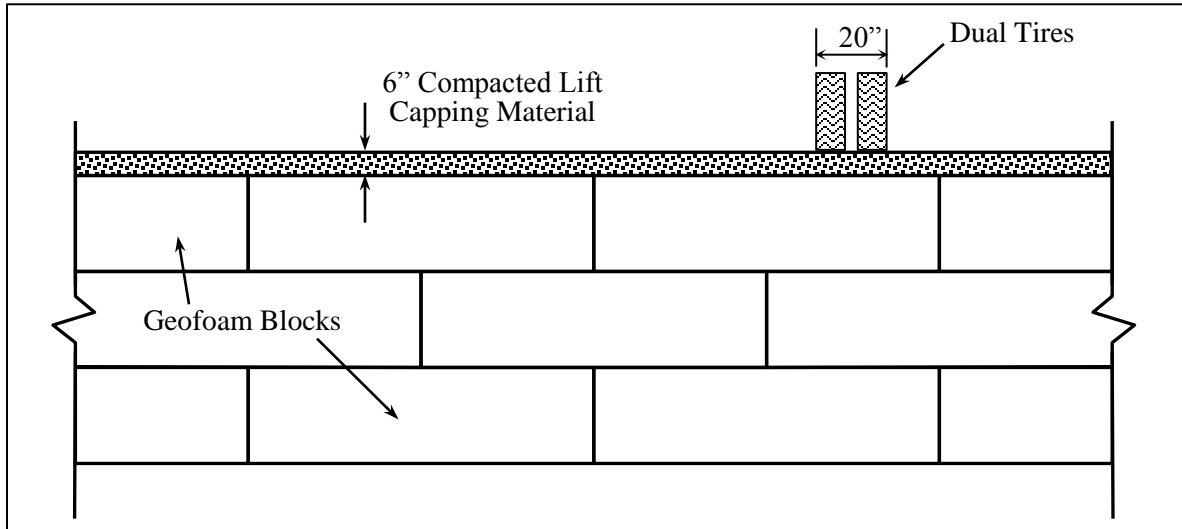


Figure 13.8.1-2 – Typical Geofoam Embankment Section after Placement of 6 Inch Capping Material

- Distribute the wheel load through the 6 inches of compacted capping material (D_c) to estimate pressure on top of geofoam block. Based on various references, pressure is mostly distributed through capping material somewhere between a 1H:2V or 1H:1V. Use 1H:2V pressure distribution as a conservative approach (refer to [Figure 13.8.1-3](#)).

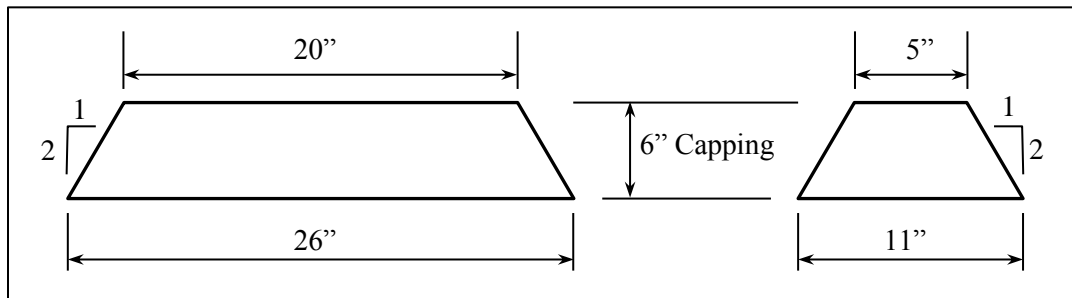


Figure 13.8.1-3 – Pressure Distribution Dimensions on Capping Material

- Determine the loaded area (A) at top of geofoam blocks (i.e., bottom of 6 inches of capping material) using the following equation:

$$A = (L)(W) = 11 \text{ in} \times 26 \text{ in} = 286 \text{ in}^2$$

- Determine the equivalent uniform pressure (P) at top of geofoam blocks using the following equation:

$$P = (\text{Distributed Pressure from Tires}) + (\text{Pressure from 6 inches Capping})$$

$$P = \frac{L_{max}}{A} + (D_c)(\gamma) = \frac{10,700 \text{ lbs}}{286 \text{ in}^2} + \frac{(0.5 \text{ ft})(130 \text{ lb/ft}^3)}{144 \frac{\text{psf}}{\text{psi}}} = 37.9 \text{ psi} \sim 38 \text{ psi}$$

Note: This is believed to be conservatively high, due to the steep 1V:2H slope assumed in the calculations for transfer of tire pressure through the capping material resulting in low distribution of the loads.

Table 13.8.1 – Geofoam Type and Properties¹

Type	Percent Deformation	Compressive Resistance
EPS22	1%	7.3 psi
EPS22	5%	16.7 psi
EPS29	1%	10.9 psi
EPS29	5%	24.7 psi
EPS39	1%	15.0 psi
EPS39	5%	35.0 psi

Note: 1. Reference: Adapted from ASTM D-6817

In conclusion, the pressure from long-term (i.e., Final Conditions) loads on the geofoam must not exceed the EPS geofoam compressive resistance at 1% deformation. However, pressure from short-term (i.e., Construction Conditions) loads on the EPS geofoam can exceed the compressive resistance at 1% deformation, but should not exceed the compressive resistance at 5% deformation. EPS39 geofoam with a compressive resistance of 35.0 psi should perform satisfactorily under construction loads due to the conservative nature of the calculations for uniform pressure which equated to approximately 38.0 psi (refer to [Table 13.8.1](#)).

To determine the required thickness of the first layer of EPS39 geofoam blocks beneath the capping material to properly distribute the stress to the underlying EPS22 geofoam:

- Assume a 1 foot thick layer of EPS39 geofoam (refer to [Figure 13.8.1-4](#)).

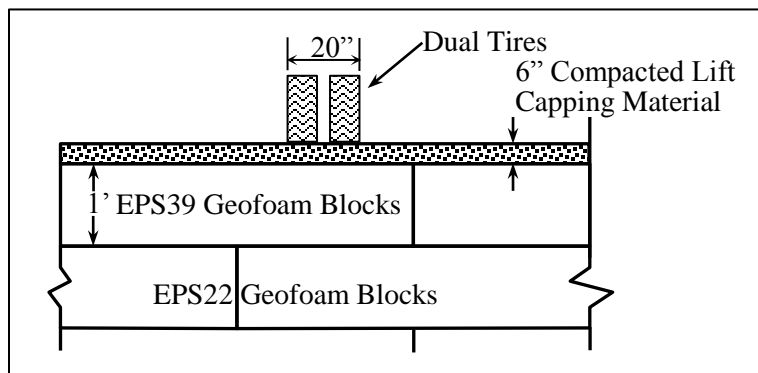


Figure 13.8.1-4 – Typical Geofoam Embankment with 1 foot EPS39 Geofoam

- Per NCHRP Web Report 65, page 6-45, vertical stress distribution through EPS geofoam can be approximated by assuming a 1H:2V distribution.
- As done in previous calculation, assume 1H:2V pressure distribution through capping material (refer to [Figure 13.8.1-5](#)).
- From page 3 of calculation, assume dual tire contact area of 20" x 5".
- Determine the loaded area (A) at top of EPS22 geofoam blocks (refer to [Figure 13.8.1-5](#)) using the following equation:

$$A = (L)(W) = 23 \text{ in} \times 38 \text{ in} = 874 \text{ in}^2$$

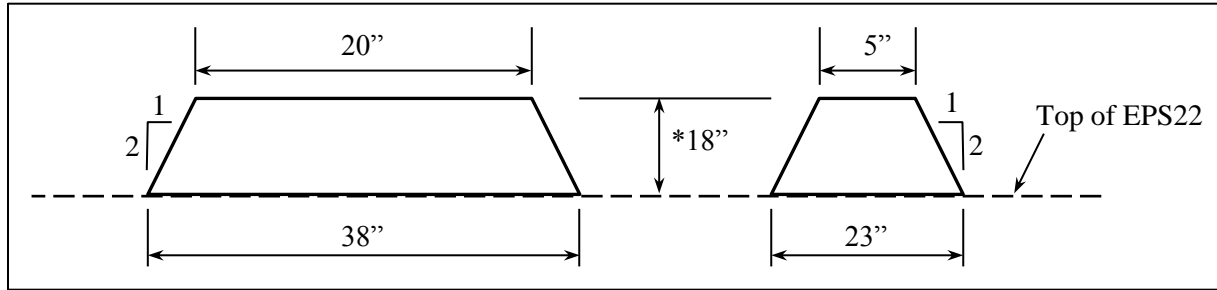


Figure 13.8.1-5 – Pressure Distribution Dimensions of EPS39 Geofoam and Capping Material

Note: The 18 inches includes 1 foot thick layer of EPS39 Geofoam with a 6-inch layer of compacted capping material.

Proceed by determining the equivalent uniform pressure (P) at the top of EPS22 geofoam blocks by using the following equation:

$$P = (\text{Distributed Pressure from Tires}) + (\text{Pressure from 6 inches Capping})$$

$$P = \frac{L_{max}}{A} + (D_c)(\gamma) = \frac{10,700 \text{ lbs}}{874 \text{ in}^2} + \frac{(0.5 \text{ ft})(130 \text{ lb/ft}^3)}{144 \frac{\text{psf}}{\text{psi}}} = 12.7 \text{ psi} \sim 13 \text{ psi}$$

In conclusion, the estimated pressure of 13 psi is below the compressive resistance of EPS22 geofoam at 5% deformation (i.e., 16.7 psi). Therefore, 1 foot of EPS39 geofoam is sufficient at the surface of the geofoam block fill, and EPS22 geofoam is acceptable to use below the EPS39 geofoam.

Proceed to check stresses for final loading conditions.

13.8.2 Final Loads/Conditions

For Final Loads/Conditions the following assumptions will be made:

- Refer to [Figure 13.8.2-1](#).
- Geofoam will be covered with a minimum of 4 feet, but not exceeding 5 feet of material. “Material” includes:
 - Capping material

- Embankment material
- Pavement section
- Use dual wheel load that was used in Construction Loads/Conditions calculations.

$$L_{max} = 10,700 \text{ lbs}/\text{set of dual tires} \quad (\text{Unless noted otherwise})$$

To calculate load factor (**IM**) in accordance with DM-4, Part B, 2015, Section 3.6.2.2 using the following equation:

$$IM = 40(1 - 0.125D_E) \geq 0\%, \text{ where}$$

IM = Dynamic Load Allowance

D_E = Minimum Depth of Cover

For D_E = 4 ft. (i.e., min. required per Publication 408, Section 219)

$$IM = 40(1 - 0.125(4 \text{ ft})) = 20\% \geq 0\% \therefore \text{OK}$$

Then, using the following equation, determine the total load (**L'_{max}**) when applying load factor (**IM**) to the wheel load:

$$L'_{max} = (1.20) (10,700 \text{ lbs}/\text{set of dual tires}) = 12,840 \text{ lbs}/\text{set of dual tires}$$

Proceed with estimating the pressure at the top of EPS39 geofoam from dual tires of a triaxle dump truck by considering the following:

- Assume a 1H:2V distribution through pavement section and embankment.

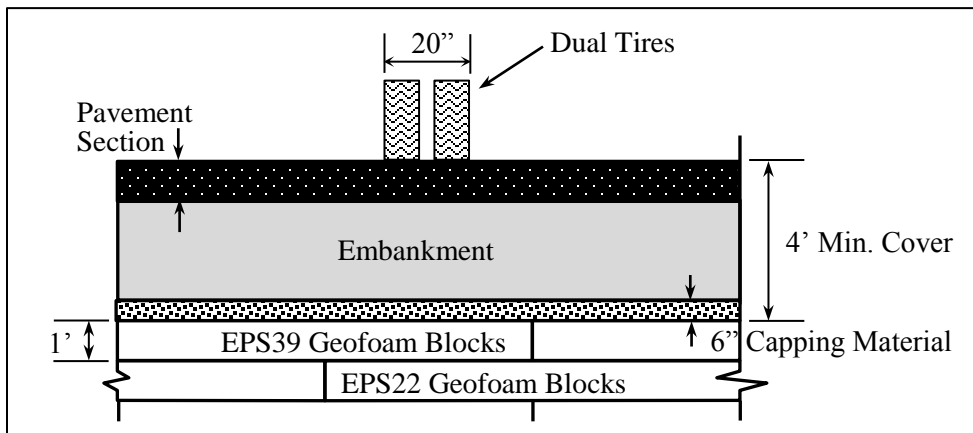


Figure 13.8.2-1 – Typical Geofoam Embankment with 4 feet of Cover

- The Distributed area (**A**) at the top of EPS39 geofoam is determined using the following equation:

$$A_1 = L + 2 \left(\frac{D_{min}}{2} \right) = 5 \text{ in} + 2 \left(\frac{48 \text{ in}}{2} \right) = 53 \text{ in}$$

$$A_2 = W + 2 \left(\frac{D_{min}}{2} \right) = 20 \text{ in} + 2 \left(\frac{48 \text{ in}}{2} \right) = 68 \text{ in}$$

$$A = (A_1)(A_2) = 53 \text{ in} \times 68 \text{ in} = 3,604 \text{ in}^2$$

- The pressure (**P**) estimated for 4 feet of cover at the top of EPS39 geofoam using factored dual axle load and pressure from pavement section, embankment, and capping material. Assume 135 pcf to account for future wearing surface overlay.

$$P = \frac{L'_{max}}{A} + (D_E)(\gamma)$$

$$= \frac{12,840 \text{ lbs/set of dual tires}}{3,604 \text{ in}^2} + \frac{(4 \text{ ft})(135 \text{ lb/ft}^3)}{144 \frac{\text{psf}}{\text{psi}}} = 7.4 \text{ psi} < 15.0 \text{ psi} \therefore \text{OK}$$

Proceed with estimating the pressure (**P**) at 1 foot below top of EPS39 geofoam (i.e., at top of EPS22 geofoam) for 4 feet of cover. Assume the dead load is relatively unchanged from 1 foot of EPS39 geofoam. Add additional pressure distribution to account for 1 foot of EPS39 geofoam.

$$P = \frac{L'_{max}}{(68 \text{ in} + 12 \text{ in})(53 \text{ in} + 12 \text{ in})} + (D_E)(\gamma)$$

$$= \frac{12,840 \text{ lbs/set of dual tires}}{5,200 \text{ in}^2} + 3.8 \text{ psi} = 6.3 \text{ psi} < 7.3 \text{ psi} \therefore \text{OK}$$

Check pressures at a maximum cover of 6 feet for final loads/conditions:

- At top of EPS39 geofoam for 6 feet of cover (pavement section, embankment & capping material):

$$\text{For IM} = 1.10, L'_{max} = 11,770 \text{ lbs/set of dual tires}$$

$$\text{For 6 feet, } A_t = 7,084 \text{ in}^2$$

$$P = \frac{L'_{max}}{A_t} + (D_E)(\gamma)$$

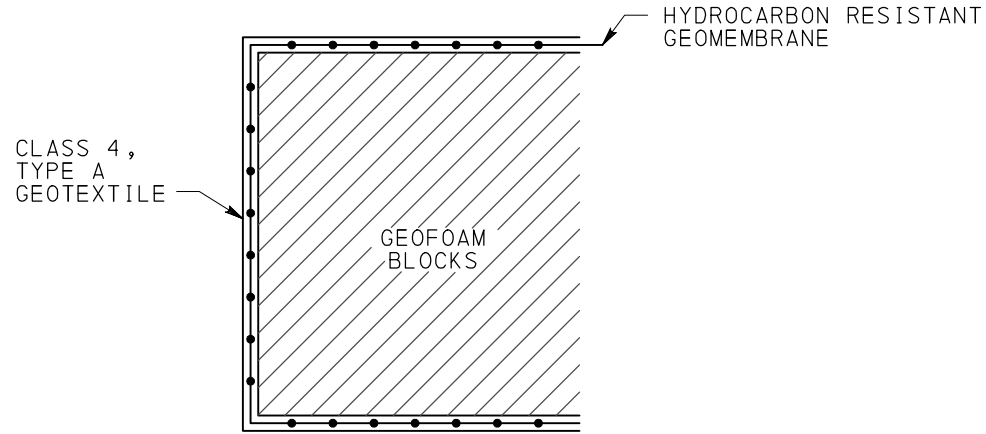
$$= \frac{11,770 \text{ lbs/set of dual tires}}{7,084 \text{ in}^2} + \frac{(6 \text{ ft})(135 \text{ lb/ft}^3)}{144 \frac{\text{psf}}{\text{psi}}} = 7.3 \text{ psi} < 15.0 \text{ psi} \therefore \text{OK}$$

- For 1 foot below top of EPS39 geofoam (i.e., at top of EPS22 geofoam) for 6 feet of cover. Assume the dead load is relatively unchanged from 1 foot of EPS39 geofoam. Add additional pressure distribution to account for 1 foot of EPS39 geofoam. Using the same factored load above:

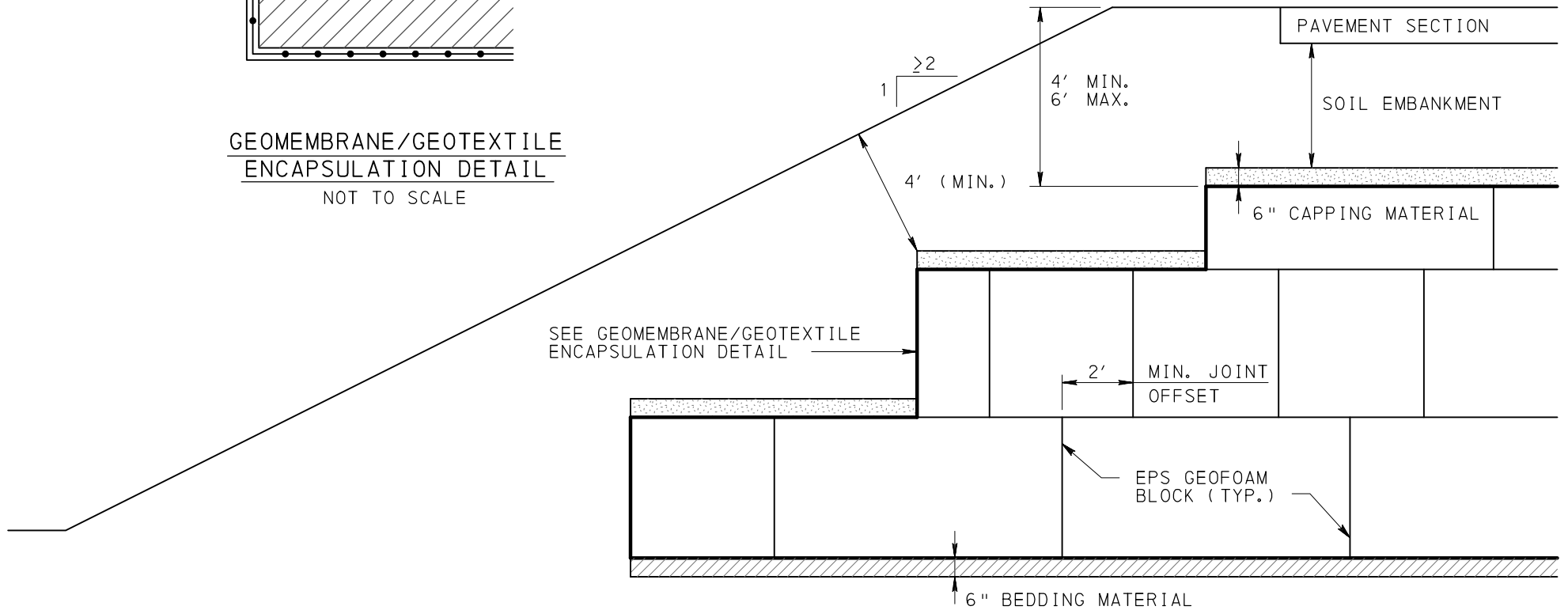
$$P = \frac{L'_{max}}{(92 \text{ in} + 12 \text{ in})(77 \text{ in} + 12 \text{ in})} + (D_E)(\gamma)$$

$$= \frac{11,770 \text{ lbs/set of dual tires}}{9,256 \text{ in}^2} + 5.6 \text{ psi} = 6.9 \text{ psi} < 7.3 \text{ psi} \therefore \text{OK}$$

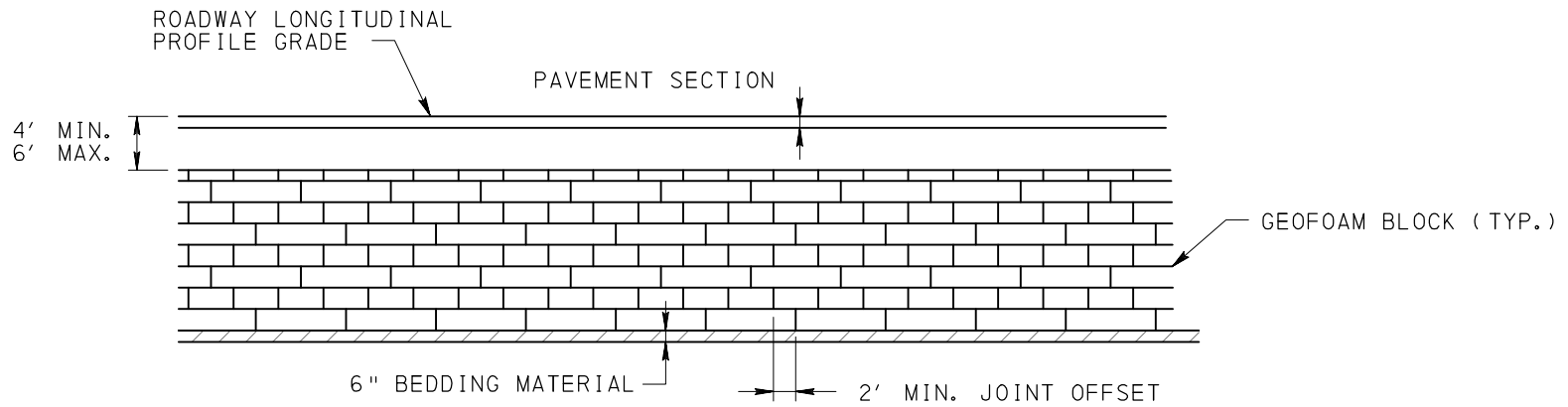
In conclusion, the estimated pressures for the final loading condition were calculated for cover (i.e., pavement section, embankment and capping material) thickness at 6 feet. Based on the calculations, the pressures at the top of the geofoam block fill (i.e., EPS39 geofoam) and 1 foot below the top (i.e., EPS22 geofoam) are below the 1% compressive resistance of the geofoam.



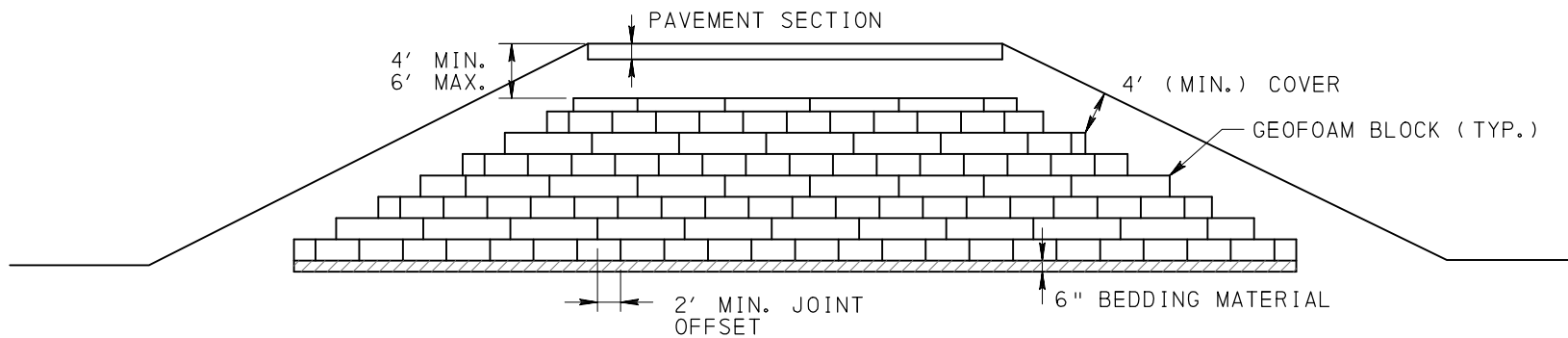
GEOMEMBRANE/GEOTEXTILE ENCAPSULATION DETAIL
NOT TO SCALE



GEFOAM SIDE SLOPE DETAIL
NOT TO SCALE



TYPICAL GEOFOAM LONGITUDINAL
PROFILE VIEW
NOT TO SCALE



NOTE:
GEOMEMBRANE/GEOTEXTILE
NOT SHOWN. SEE DETAIL.

TYPICAL GEOFOAM
EMBANKMENT SECTION
NOT TO SCALE

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GEOTECHNICAL ENGINEERING MANUAL**

CHAPTER 14 – GEOSYNTHETICS USE GUIDELINES

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2018
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 15 – ROCK SLAKE DURABILITY

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15.1 INTRODUCTION

This chapter provides: standard procedures to identify where slaking rock exists; guidance to assess the slake durability of these rocks through simple test methods; direction for the interpretation of the test results; and discussion of appropriate mitigation strategies for the design of structural elements such as spread footings, driven piles, and drilled shafts in slake-prone, low-durability rock.

15.1.1 Purpose

Fine-grained sedimentary rocks which include shales, claystones, mudstones, and siltstones are very common within the surface/near-surface geology of Pennsylvania and are frequently encountered during highway design and construction. Some of these rock types, when exposed to air and water, can begin to slake or disintegrate. These slaking or low-durability rocks can present adverse geotechnical conditions during highway and bridge design and construction. Consequently, clear guidelines which assist the designer in identification of these materials, provide testing procedures, and present a method for the interpretation of results to support geotechnical highway design are necessary. These guidelines are presented in the subsequent sections.

15.1.2 Terms and Definitions

The following definitions are commonly associated with evaluating the slake durability of fine-grained sedimentary rock types.










1. **Cementation** – Process by which dissolved minerals become deposited in the spaces between individual sediment grains. These dissolved minerals act as a cement to bind sediments and mineral particles together thereby lithifying the sediments into rock.
2. **Compaction** – The densification of soil particles by application of static or dynamic pressure that results in an increased unit density due to a reduction of inter-particle air voids.
3. **Durability** – The ability of the material mass to resist physical breakdown into smaller aggregate sizes, particularly due to the effects of abrasion, cyclical freezing and thawing, and cyclical wetting and drying.
4. **Fissility** – The property of sedimentary rock to split easily along planes of weakness into thin layers along closely spaced, parallel surfaces.
5. **Lithification** – The conversion of unconsolidated sediments to rock by three primary processes: compaction, cementation and recrystallization.


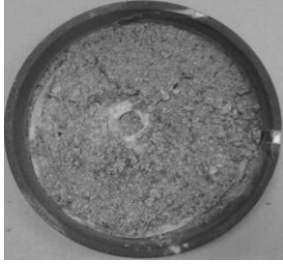
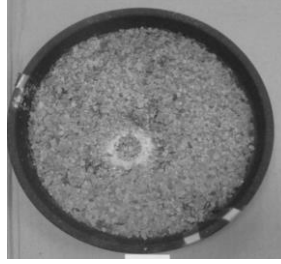
6. **Slaking** – The disintegration of rock under wetting and drying conditions and when exposed to air. This behavior is related primarily to the chemical composition of the rock.

15.1.3 Overview of Slaking

Fine-grained sedimentary rock which remains underground, confined and undisturbed, maintains its inherent composition and strength, even when natural moisture conditions fluctuate. When some fine-grained sedimentary rock types are excavated and exposed, subsequent fluctuations in moisture content can begin to cause degradation. This degradation is generally termed “slaking”. These rocks can quickly degrade from an apparent hard, blocky, rock mass into some combination of fractured rock pieces, rock chips, thin rock flakes, granular mineral particles, and/or mud paste. Depending on the susceptibility of the rock to slaking, the breakdown can occur within minutes of exposure, or could happen progressively over a period of days, weeks, months, or years as various cyclical environmental stressors act upon the exposed rock. [Table 15.1.3-1](#) shows results of weathering of intact rock samples in the natural environment that were part of a research study conducted in Ohio (Gautam, T.P., An Investigation of Disintegration Behavior of Mudrocks Based on Laboratory and Field Tests, 2012).

Table 15.1.3-1 – Natural Weathering of Fine-grained Sedimentary Rock Samples¹

Elapsed Time ²	Siltstone	Shale	Claystone
Initial Sample condition			
After 1-month of weathering			
After 6-months of weathering			

Elapsed Time ²	Siltstone	Shale	Claystone
After 12-months of weathering			

Notes: 1. Reference: Gautam, 2012

2. Not all claystone, siltstone, and shale deposits produce the same type and magnitude of slaking.

The extent of slaking or degradation shown in [Table 15.1.3-1](#) is not guaranteed for a given rock type, and is more variable in shales than claystones and siltstones. The severity of slaking is dependent upon the individual rock type and environmental conditions. The slake durability or resistance to softening or flaking of a given sedimentary rock is to a large extent dependent upon its degree of lithification. The type of clay mineral and cement bonding that binds the silt and clay particles together are also key factors in the resistance to slaking exhibited by a given rock.

Physical weathering conditions in Pennsylvania include: wetting-drying cycles, freeze-thaw cycles, and heating-cooling cycles. Each of these cycles can serve to breakdown rock material with time by exerting various internal stresses near exposed surfaces of rock. It should be recognized that while each form of physical weathering tends to degrade intact rock and reduce the overall rock strength, that strictly speaking, slaking is a result of the moisture fluctuations and exposure to air. Evaluating the effect of wetting-drying cycles is the focus of this chapter.

15.2 CONDITIONS WHERE SLAKING MAY AFFECT FOUNDATION DESIGN

Slaking rock conditions can have negative impacts on both deep and shallow foundations. Deterioration of rock due to slaking can cause severe reductions in shear strength (Schaefer, V.R., Mechanisms of Strength Loss During Wetting and Drying of Pierre Shale, 2013). Two important considerations are the degree of slake deterioration and the proximity of the suspected slaking rock to the foundation elements. For example, if slaking rock is providing direct support to driven piles, moderate or severe slaking can result in significant reduction of pile resistance. Therefore, any suspect slaking rock that will provide geotechnical resistance for a structure foundation must be evaluated for slake potential.

[Table 15.2-1](#) provides a list of commonly used foundations for Department projects, and a list of concerns if slaking rock is identified within the bearing material. [Section 15.5](#), Mitigation Strategies and Foundation Design Recommendations for Slaking Rock, provides guidance on mitigation strategies for handling slaking rock during foundation design.

Table 15.2-1 Geotechnical Concern for Foundation Design in Slaking Rock

Foundation Type	Geotechnical Concern
Spread Footings	Softening of slaking rock due to exposure to air and moisture during construction, resulting in reduction of bearing resistance.
Driven Piles	Driven pile creating pathway for water to reach slaking rock, resulting in softening of rock and reduction of axial and/or lateral resistance.
Micropiles and Drilled Shafts	Softening of slaking rock due to exposure to air and moisture during construction, resulting in reduction of axial and/or lateral resistance.

15.2.1 Spread Footings

Slaking presents two primary risks to spread footings that bear directly on rock; the loss of shear strength and reduced resistance to scour. This loss of strength increases the risk of bearing and/or sliding failure. Footings in open water environments can be vulnerable to erosion/scour if the slaking potential of the founding rock is determined to be high. Slaking rock can become significantly less resistant to the erosive energy of flowing water, and any such bearing strata that is exposed to stream flow can lead to eventual undercutting of the foundation.

15.2.2 Driven Piles

End-bearing piles that are driven into slaking rock may relax and may not maintain their axial resistance obtained during initial driving. This relaxation can be a result of subsequent rock softening and/or the dissipation of negative pore pressures (suction) developed during driving. The disturbance and breakage of the rock caused by driving the pile reduces the rock confinement pressure. Additionally, pile driving can create a pathway for water to reach the pile tip, facilitating slaking.

15.2.3 Micropiles and Drilled Shafts

It is generally not acceptable to bear micropiles or drilled shafts in slaking rock because they may fail to maintain their axial and/or lateral load resistance. Some exceptions for drilled shafts include lightly loaded foundations for light poles or retaining structures.

15.2.4 Weathering of Rock Cut Slopes

[Chapter 8](#) of this publication, titled Rock Cut Slope and Catchment Design, provides guidelines, recommendations, and considerations for the design of new rock cut slopes and rehabilitation of existing rock cut slopes. [Chapter 8](#) includes discussion of slaking rocks with respect to rock cuts.

15.3 IDENTIFYING SLAKING ROCK

It is necessary to identify rock types that are possibly slake prone for any project requiring structure foundations. The sequence of methods to identify suspected slaking rock is, as follows:

1. Locate the project on geologic mapping that may indicate weak sedimentary rock types.
2. Check for published geotechnical documentation of known slake-prone geologic formations of concern.
3. Examination of the rock core samples by the Project Geotechnical Engineer to determine the general rock composition, apparent hardness, and fissility, and to assess the need for slake testing.
4. Perform laboratory testing, if deemed appropriate.

These individual methods are described in further detail in the following subsections.

15.3.1 Stratigraphy and Geologic Mapping

Initial identification of potentially slaking rock should begin with consulting the most detailed published geologic map and stratigraphic columns available for the project area, and determining the geologic unit or units underlying the project area. If the geologic map indicates the project site is underlain by limestone, sandstone, igneous or metamorphic rock, concerns for slaking are minimal and may often be eliminated.

A majority of slaking rock formations in Pennsylvania were formed around the Pennsylvanian and Permian time period, and are located within the Appalachian Plateaus physiographic province. This province is dominated by alternating sedimentary rock sequences of shale, siltstone, coal, claystone, mudstone, limestone, and sandstone. These fine-grained sedimentary rocks (i.e., shale, siltstone, claystone, and mudstone) are less indurated (hardened), and more likely to slake and disintegrate when exposed to air and water. [Table 15.3.1-1](#) lists geologic units which are well known and documented to contain slaking rocks. [Table 15.3.1-1](#) is not an all-inclusive list of slaking rocks located within Pennsylvania but is intended to preliminarily alert the designer of known stratigraphic units which have a high potential for slaking. For each project, the drilling inspection logs and rock core samples must be carefully examined and considered for suspected slaking rock types. An attempt should be made to correlate the rock core samples to published stratigraphic maps or geologic studies of the area. Any potential slaking stratigraphic zones that may have a potential impact on project design should be examined closely and targeted for laboratory testing.

Table 15.3.1-1 Slake-Prone Geologic Units in Pennsylvania

Geologic Group	Geologic Formation¹	Slaking Potential²
Dunkard	Greene	Moderately High to Severe
	Washington	
	Waynesburg	Moderately High
Monongahela	Uniontown	Moderately High
	Pittsburgh	
Conemaugh	Casselman	Moderately High to Severe, various red beds, including the “Clarksburg” and “Schenley”
	Glenshaw	Moderately High to Severe “Pittsburgh red beds”
-	Allegheny	Low to Moderate
	Pottsville	Low

- Notes: 1. Table is not inclusive of all geologic formations having slaking rock.
2. Conditions are typical and may vary.

The central and eastern portions of Pennsylvania, more specifically the Ridge and Valley, Piedmont, and New England physiographic provinces are comprised of sedimentary, igneous, and metamorphic rock types as shown in [Figure 15.3.2-1](#). Shale and siltstone units are present within these physiographic regions, however, they are generally more indurated and less slake prone than similar lithologies located in the Appalachian Plateaus Province.

The presence of slaking rock formations often coincides with high occurrences of landslides. As shown in [Figure 15.3.2-1](#), the Waynesburg Hills Section and the Pittsburgh Low Plateau Section of the Appalachian Plateaus Province, have the highest landslide susceptibility within Pennsylvania. Consequently, slaking, sedimentary rock sequences are also prevalent within these areas.

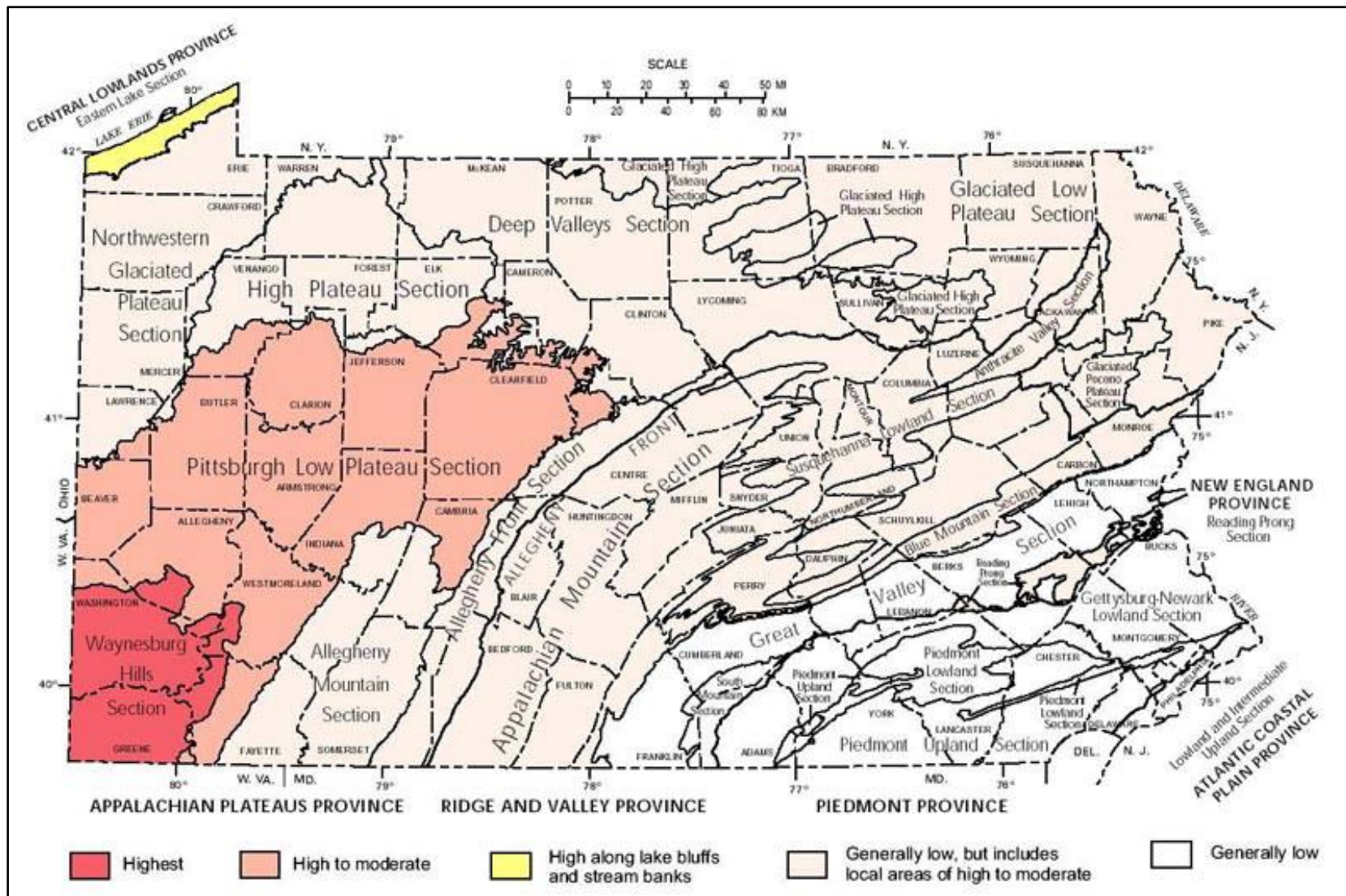


Figure 15.3.2-1 – Landslide Susceptibility Map of PA (PA-DCNR)

15.3.2 Rock Identification and Description

Rock type, rock composition, rock hardness, and rock fissility are four primary rock characteristics that are used to assist in anticipating slake susceptibility. They are discussed in the following subsections in more detail.

15.3.2.1 Rock Type

The slake-prone rock types listed in [Table 15.3.2.1-1](#) are based on characteristics such as predominant grain-size and laminations.

Table 15.3.2.1-1 Slake Potential based on Rock Type

Rock Type	Description	Slake Potential
Claystone	A fine-grained sedimentary rock formed of predominantly clay-sized particles. Claystone is comprised of lithified clay possessing the texture of shale but lacks laminations and fissility of a shale. Exhibits a massive, blocky appearance and often possesses slickensides.	Generally exhibits low slake durability (i.e., high slaking potential)
Siltstone	A fine-grained sedimentary rock formed of predominantly silt-sized particles. Siltstone is comprised of lithified silt and lacks lamination and fissility. Exhibits a fine gritty texture.	Generally exhibits medium slake durability (i.e., moderate slaking potential)
Shale	A fine-grained sedimentary rock formed of clay, or clay-sized and silt-sized particles. Shale exhibits a laminated structure which gives it fissility along which the rock readily splits.	Generally exhibits low to medium slake durability (i.e., moderate to high slaking potential)

15.3.2.2 Rock Composition

Rock composition is an important factor in slake assessment, but it cannot be used alone to accurately predict material behavior. Although clay is known to be the main constituent of many slaking rock types, the amount of clay, type of clay, degree of compaction, cementation, fracturing, and organic content all have a potential influence on the physical properties of the rock. A relatively minor amount of clay content can have a significant effect on the slaking behavior of the rock. If the rock sample has a clayey feel, it likely has ample clay mineral content to be considered a highly suspect slaking rock. Regardless of the perceived clay content by visual inspection, any rock that has the potential for slaking should be considered for slake testing.

Additional rock composition properties that should be considered are unit weight and natural moisture content. Research by T. Masada (Rock Mass Classification System: Transition from RMR to GSI, 2013) and ODOT (Rock Slope Design Guide, 2011) suggests rock having a

unit weight less than 140 pcf has been shown to commonly have a low slake durability. Conversely, rock having a unit weight greater than 160 pcf has been shown to commonly have a high slake durability. Research by L.S. Bryson (Correlation between Durability and Geotechnical Properties of Compacted Shales, 2011) suggests that rock deposits having a natural moisture content below 4% were expected to be slake resistant with a corresponding Slake Durability Index value greater than 85%.

Laboratory determination of unit weight and natural moisture content is discretionary, but recommended, for the foundation design. These values may assist in determining the extent of slake testing needed, and also confirm the overall results. If this information is obtained, it should be used in conjunction with the slake durability test results to assist in the evaluation of the anticipated slaking performance of the foundation rock. When testing the natural moisture content of a rock sample, field preservation of the rock's natural moisture content is important, and should be completed immediately upon removal of the rock from the core barrel.

15.3.2.3 Rock Hardness

The apparent hardness of a freshly cored rock sample is a material characteristic that is always determined and documented by the PennDOT Certified Drilling Inspector. The apparent hardness could be misleading if the possibility of rock softening over time due to slaking is not considered. This is the underlying basis for completing the slake testing. Determining the apparent hardness of a rock sample is used to give an initial indication of possible slake susceptibility and helps to determine which rock interval should be selected for further testing. Slake-prone rock tends to be weakly bonded due to weathering, weak cements, and/or significant clay content, and will often appear less hard than a rock type that is bonded by harder mineral cement types. Rock identified as 'very soft' or 'soft' is generally much more suspect to slaking than a 'medium hard' rock. Sedimentary rock types identified as 'hard' or 'very hard' likely contain durable mineral cements and are typically much less prone to slaking when exposed to weathering.

15.3.2.4 Rock Fissility

Any rock type identified by the drilling inspector as shale or described as 'shaley' is expected to be fissile to some degree, which by definition splits along planes of weakness. It is possible for a rock to have visual laminations but not physically split easily or weakly along laminations.

Siltstones and claystones that are stratified may be visually similar to fissile rock types (shales), but may differ in percent silt and clay composition, and in how readily the laminations tend to part or split. When the strength of the lamination bonds are equal to that of the rock layers, the horizontal and vertical shear strength are equal (i.e., the rock is isotropic). In this case, the rock is not fissile, and is likely not shale. When a rock sample has closely spaced lamination interfaces that are only slightly weaker than the rock layers themselves, the layers can be mechanically broken into closely spaced, parallel layers, and the rock could be identified as 'shaley' or 'fissile'.

Since rock fissility enhances the potential surface area of the rock mass that is exposed to weathering effects (air and water), the intensity of the fissility is thought to have some correlation to the degree of slake potential (i.e., the more fissile the rock, the more likely it is to slake).

15.3.3 Laboratory Testing

Laboratory testing is the final step in the identification and confirmation process of slaking rock. This step is normally completed only if the previous steps (i.e., mapping, stratigraphy, rock identification/description) indicate that the rock may be slake prone. However, laboratory testing can be conducted as a precautionary measure if there is doubt or concern regarding the slake potential of the rock.

There are two different slake index tests used by the Department to assist in qualitatively and quantitatively assessing the slaking behavior of rock: the Jar Slake (Index) test and the Slake Durability (Index) test. The description and use of these tests are given in [Section 15.4](#).

15.3.4 Selecting Samples for Laboratory Testing

Rock samples obtained from stratum that may be used to provide resistance for shallow and deep foundations, and that are potentially susceptible to slaking (e.g., as determined by previously discussed methods) must be tested in the laboratory. A minimum of two rock samples should be tested from each substructure and from each stratum that is potentially slake prone for the Jar Slake Test. Per ASTM D4644, ten samples are required for the Slake Durability Test. Only stratums that are anticipated to provide foundation resistance need to be tested. [Figure 15.3.4-1](#) provides some examples of how to select rock samples for testing based on stratigraphy and foundation type. These are only guidelines, and testing requirements will vary based on numerous factors, including: project size and complexity; proposed construction/foundation type; and rock type(s), stratigraphy, thickness, and consistency.

The samples should first be tested using the Jar Slake Test method. Based on the results of the Jar Slake Test, a minimum of two additional samples may need to be tested using the Slake Durability Test method. Guidelines to determine when it is necessary to perform the Slake Durability Test are provided in [Section 15.4.1](#).

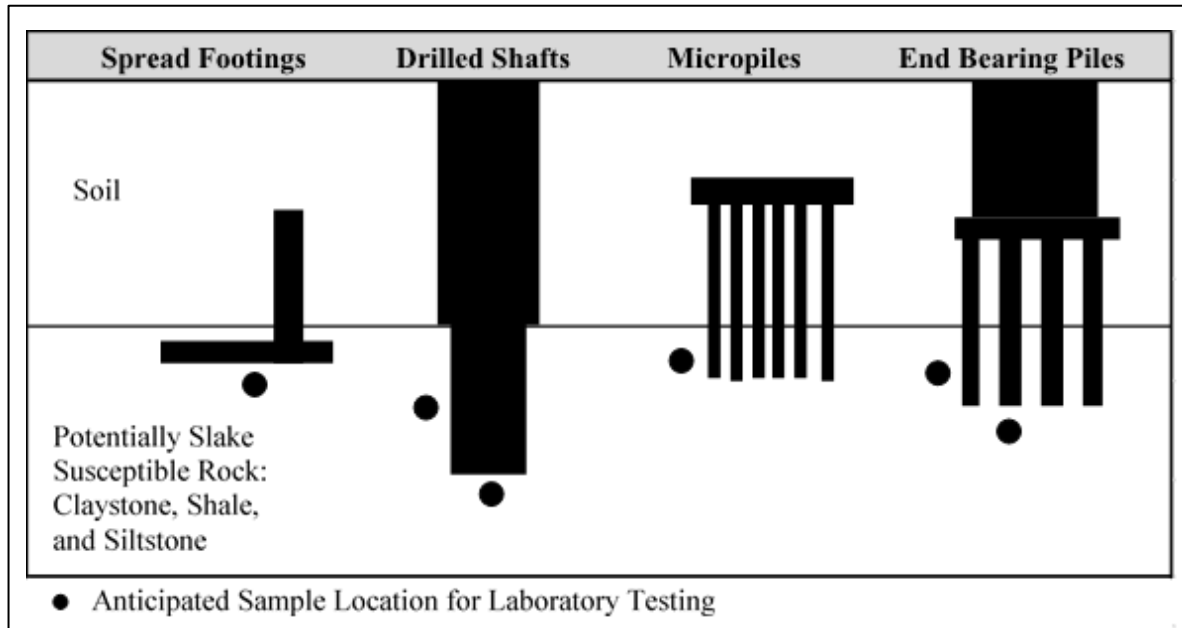


Figure 15.3.4-1- Guidelines for Laboratory Sample Location Selection

15.4 SLAKE TESTING METHODS, RESULTS AND INTERPRETATION

This section provides requirements for slake testing and guidance for the interpretation of laboratory slake testing results. Interpretations include the rock's anticipated geotechnical load bearing performance, and if supplemental laboratory testing is required.

As described by D.N. Richardson (Shale Durability Rating System Based on Loss of Strength, 1990) well over 20 slake classification systems, involving over 50 types of laboratory tests/methods, have been developed. A common deficiency with these methods is the lack of direct correlation between classification or index test values and fundamental material properties used for foundation design (Chapman, D.R., A Comparative Study of Shale Classification Tests and Systems, 1976). The type and magnitude of slake stressors applied by the laboratory testing (short-term) are likely different than the actual field conditions (long-term). Despite this deficiency, empirical correlations of laboratory slake index values remains the current standard of practice for foundation design.

15.4.1 Jar Slake Test






The Jar Slake Test is a relatively simple, inexpensive test that is intended to quickly assess the overall slake potential of rock. This test can conclusively identify rocks that are significantly prone to slaking (i.e., low slake durability). This test is also used to determine if more detailed testing, specifically Slake Durability Testing discussed in [Section 15.4.2](#), is needed. The Jar Slake Test is not effective for estimating the slake potential of medium to high durability rock.





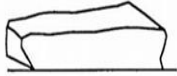

The Jar Slake Test method that is to be used on Department projects is detailed in PTM No. 122. This PTM closely follows the jar slake test method developed by Deo (Use of Shale in

Embankments, 1972). The test is done by placing a 100 to 200-gram oven-dried sample in water and observing the sample at specified intervals over a 24-hour period. It is important that oven-dried samples be used because it has been reported that material tested at its natural moisture content is notably less sensitive to degradation compared with like samples that have been tested after being oven-dried (Caltrans, Soil and Rock Logging, Classification, and Presentation Manual, 2007).

[Table 15.4.1-1](#) provides guidelines for interpreting Jar Slake Test results. The table correlates the observations made during the Jar Slake Test to a Jar Slake Index (IJ) value ranging from 1 to 6. The table further provides interpretation of the slake potential of the sample tested, and if more rigorous testing (i.e., Slake Durability testing per ASTM D4644) is required.

Table 15.4.1-1- Jar Slake Test Interpretation Guideline

Observations of Rock Sample During Jar Slake Test	Example Photograph	I _J	Material Interpretations	Further Slake Durability Testing Required?
<p>Rapid breakdown (several minutes or hours) of entire sample into fine flakes and/or fine sediment (mud).</p>  <p>1. MUD - degrades to a mud-like consistency.</p>		1	<p>Slaking Material: Rock is most certainly slake prone. If rock/stratum is used to provide foundation resistance mitigation is required. Material interpretation the same for I_J values of 1 and 2.</p>	No.
<p>Breakdown of majority of sample into fine sediment and flakes.</p>  <p>2. FLAKES - sample totally reduced to flakes. Original outline of sample not discernible.</p>		2		
<p>Breakdown of sample into separate rock fragments (fracturing) with significant surface slaking (softening).</p>  <p>3. CHIPS - chips of material fall from the sides of the sample. Sample may also be fractured. Original outline of sample is barely discernible.</p>		3	Same as material interpretation for I _J values of 1 and 2.	No.

Observations of Rock Sample During Jar Slake Test	Example Photograph	I _J	Material Interpretations	Further Slake Durability Testing Required?
<p>Breakdown into rock fragments (fracturing) with minor surface slaking (softening). Soak water will be cloudy when jar is rotated.</p>  <p>4. FRACTURES - sample fractures throughout, creating a chunky appearance.</p>		4	<p>Likely Slaking Material: Rock is likely slake prone. May not slake under short-term exposure. May require mitigation if used to provide foundation resistance.</p>	<p>Yes. Proceed to 15.4.2.</p>
<p>Very minor fracturing of sample is evident with no surface slaking (softening) after 24-hour soak. Soak water remains clear or nearly clear.</p>  <p>5. SLABS - sample parts along a few planar surfaces.</p>		5	<p>Possibly Slaking Material: Rock is possibly slake prone. May require mitigation if used to provide foundation resistance.</p>	<p>Yes. Proceed to 15.4.2.</p>
<p>No change to condition of the rock sample discernible after 24-hour soak. Soak water remains clear.</p>  <p>6. NO REACTION - no discernible effect.</p>		6		

15.4.2 Slake Durability Test

If the results of the Jar Slake Test do not conclusively indicate that the rock is slake prone (i.e., I_J value of 4 to 6), more rigorous testing using the Slake Durability Test method per ASTM D4644 is required. Similar to the Jar Slake Test, the Slake Durability Test is an empirical test that is used to estimate the long-term performance of the rock based on results of a short-term

laboratory test. The main difference between the two test methods is the rock sample is subjected to more and longer disturbance during the Slake Durability Test. Although the Slake Durability Test was originally developed for testing shales, any potentially slaking rock can be tested.

A brief outline of the Slake Durability Test (ASTM D4644) is as follows. Refer to the actual ASTM test document for the detailed requirements of the test.

1. Oven-dry a rock sample weighing approximately 500 grams
2. Place sample in a testing drum, and rotate the drum in water for 200 revolutions.
3. Remove the sample from the drum and oven-dry.
4. Repeat Steps 2 and 3. The results of the test are expressed as the percentage of dry weight of rock retained following second cycle, ID(2).

$$I_{D(2)} = \left(\frac{\text{Dry Weight Retained (after two cycles)}}{\text{Dry Weight (before test)}} \right) \times 100\%$$

As discussed in [Section 15.3.2.2](#), rock composition (type) is to be considered as part of the assessment of slake durability. [Table 15.4.2-1](#) presents some anticipated slake durability index ranges for various slake-prone rock types that are commonly encountered in Pennsylvania. The information presented is to be used to help assess the validity of the actual laboratory test results conducted for the project. **This information is not to be used as a substitute for laboratory testing.**

Table 15.4.2-1- Typical Range of Slake Durability ID(2) Based on Rock Type^{1,2}

Rock Type	Typical Dry Unit Weight, pcf	Typical Slake Durability Index, I _{D(2)} , %
	Range	Range
Siltstone	145-165	65-90
Shale	125-160	20-90
Claystone	115-155	0-60

Notes: 1. Reference: Masada, 2013; Geyer, A.R., (Engineering Characteristics of the Rocks of Pennsylvania 1982); Underwood, L.B., (Classification and Identification of Shales, 1967); et al..

2. Values can differ significantly for a given rock type.

The durability classification proposed by J.C. Gamble (Durability-Plasticity Classification of Shales and Other Argillaceous Rocks, 1971) and recommended by the International Society for Rock Mechanics in 1979 is based on the ID(2) value. This classification has six classes of durability, which include: very high (100-98%), high (98-95%), medium high

(95-85%), medium (85-60%), low (60-30%), and very low (30-0%). J.C. Dick (A Geological Approach Toward Developing a Mudrock Durability Classification, 1994) proposed a durability classification which looked at ID(2) values in conjunction with lithologic characteristics such as clay content, dry density, and microfracturing. This classification categorizes all claystone as low-durability (ID(2) <50%) whereas, siltstones, and shales can occur in one of the three classes: low (ID(2) <50%), medium (ID(2) 50% - 85%), and high (ID(2) >85%). The classification proposed in the Gautam (2012) study is based on a calculated “disintegration ratio” in an effort to account for the variation in size of fragments produced during the slake durability test.

[Figure 15.4.2-1](#) shows a comparison of the slake durability classifications given by Gamble, Dick, and Gautam to demonstrate the relative consistency of the conclusions that have been presented by various researchers over a period of several decades. In general, rock having a slake durability index above approximately 85% is considered durable, while material having a value below approximately 60% is considered non-durable.

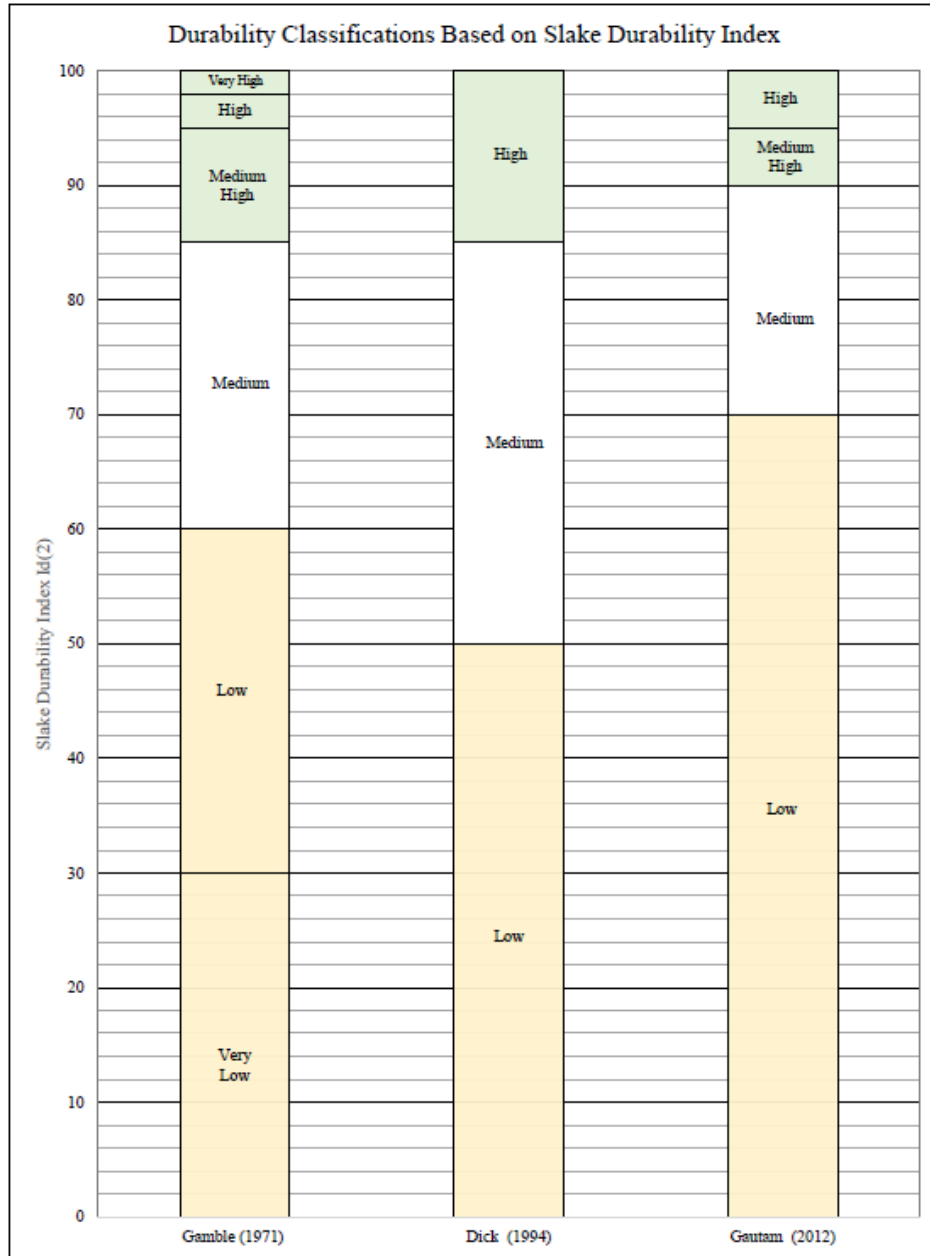


Figure 15.4.2-1- Comparison of Published Slake Durability Classifications

[Table 15.4.2-2](#) provides the Department’s general interpretations of the Slake Durability Test results. These general interpretations simply indicate the anticipated durability of the samples tested.

Table 15.4.2-2- Slake Durability Test General Interpretations

I _{D(2)}	Material Interpretations, ASTM D4644	Material Loss
< 60%	Low slake durability. The highest suspect rock types (typically claystone) will be in this range. Material can be expected to revert to soil.	> 40%
60% – 85%	Medium slake durability. When sufficiently exposed to drying and cyclical moisture conditions, material can be expected to revert to an intermediate mix of rock and soil. Subsequent decomposition yields a matrix of soil particles, gravel particles, and intact pieces of rock. Likely maintains a sufficient rock fraction to provide higher shear strength over a soil alone. I _{D(2)} test results having significant rock fragments remaining suggest a joint or fracture softened condition.	15 – 40%
> 85%	High slake durability. Material exposed to drying and cyclical moisture conditions can be anticipated to maintain rock structure and performance in long-term service life. In this range, no additional requirements or restrictions are needed relative to slaking potential for materials.	< 15%

These general interpretations are anticipated to be applicable to a majority of Department projects based on typical rocks encountered in Pennsylvania. It is recognized and emphasized that atypical and unique conditions do exist and will be encountered on some projects. Engineering judgement must always be used to identify and address uncommon conditions when they occur.

15.5 FOUNDATION DESIGN GUIDELINES FOR SLAKING ROCK

There are no industry standard foundation design recommendations based on laboratory slake test results. Therefore, designers must consider the slake test results in combination with the other rock properties discussed in this chapter, engineering judgment, and past experience when designing a foundation in slake-prone rock. District Geotechnical Engineers are encouraged to consult with the Chief Geotechnical Engineer when projects involve foundations in or in proximity to low slake durability rock.

Tables [15.5-1](#) and [15.5-2](#) provide the Department’s general foundation design recommendations based on the Jar Slake Test and the Slake Durability Test.

Table 15.5-1 – Foundation Design Recommendations Based on Jar Slake Test Results

Jar Slake Index, I_J	Spread Footings	Driven Piles	Micropiles and Drilled Shafts
1 to 3	Do not place spread footings directly on low-durability rock. Place a 6-inch layer of Class C concrete over bedrock foundation immediately upon excavation / foundation approval. Design per DM-4 based on results of unconfined compression tests on intact rock cores, or as an equivalent soil mass.	Do not terminate driven piles in low-durability rock. If piles cannot be driven through slake prone rock, predrilling will be needed. DM-4 D10.4.7.2.4P indicates predrilling is recommended for penetrating claystone layers 2 feet or more in thickness.	Do not terminate drilled shafts or micropiles in low-durability rock. Intervals of slaking rock are to be ignored when calculating shaft resistance.
4 to 6	See Table 15.5-2 for foundation design recommendations.		

Table 15.5-2 – Foundation Design Recommendations Based on Slake Durability Test

Slake Durability Index, 2 nd Cycle, $I_{D(2)}$	Spread Footings	Driven Piles	Micropiles and Drilled Shafts
< 60%	Do not place spread footings directly on low-durability rock. Place a 6-inch layer of Class C concrete over bedrock foundation immediately upon approval of the footing excavation. Design per DM-4 based on results of unconfined compression tests on intact rock cores or as an equivalent soil mass.	Do not terminate driven piles in low-durability rock. If piles cannot be driven through low-durability rock, predrilling will be needed. Piles can typically be driven through thin layers of weak, low-durability rock. DM-4 D10.4.7.2.4P indicates predrilling is recommended for penetrating claystone layers 2 feet or more in thickness.	Do not terminate drilled shafts or micropiles in low-durability rock. Intervals of slaking rock are to be ignored when calculating shaft resistance.

Slake Durability Index, 2 nd Cycle, I _{D(2)}	Spread Footings	Driven Piles	Micropiles and Drilled Shafts
60% – 85%	Do not place spread footings directly on medium-durability rock. Place a 6-inch layer of Class C concrete over bedrock foundation immediately upon approval of the footing excavation. Design per DM-4 based on results of unconfined compression tests on intact rock cores or as an equivalent soil mass.	Specify a restrike and the use of dynamic monitoring for driven piles with potential to terminate in medium-durability rock in order to assess the effect of relaxation or setup and confirm pile capacity.	Do not terminate drilled shafts or micropiles in medium-durability rock unless they are supporting lightly-loaded structures, such as sound barriers, high mast lights, certain ITS equipment, etc.
> 85%	No consideration for slake is needed for design or construction on high-durability rock.		

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APPENDIX B – CLAY AND IMPERVIOUS LINERS

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APPENDIX C – COMPACTION GROUTING

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APPENDIX D – ROCK MASS RATING (RMR)**

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APPENDIX G – SUBSIDENCE