OS-299 (10-22)

pennsylvania
DEPARTMENT OF TRANSPORTATION
www.penndot.pa.gov

TRANSMITTAL LETTER

PUBLICATION:

Publication 293

DATE:

12/14/2022

SUBJECT:

December 2022 Edition, Publication 293 - Geotechnical Engineering Manual

INFORMATION AND SPECIAL INSTRUCTIONS:

Publication 293 (Geotechnical Engineering Manual), December 2022 Edition is to be issued with this letter.

Chapter 6, Soil Slope Stability, and Chapter 7, Settlement, of this manual were completed by subject matter experts and circulated for review and comments through the Clearance Transmittal process on CT# C-22-003 and CT# C-22-005, respectively. This release incorporates Chapter 6 and 7 into this edition of Publication 293. Clarification was added to Chapter 9 of this publication regarding compaction for bridge embankments. Updates to Chapter 4 to address sample size were completed.

Also, included throughout are updates or revisions that have been implemented via Strike-Off Letter or are editorial in nature. Chapter 11, Geosynthetic Reinforced Soil (GRS) Slopes, of this manual was revised by subject matter experts and circulated for review and comments through the Clearance Transmittal process on CT# H-21-030. This release incorporates the revisions of Chapter 11 into this edition of Publication.

CANCEL AND DESTROY THE FOLLOWING:

Publication 293, December 2020 Edition

ADDITIONAL COPIES ARE AVAILABLE FROM:

☒ PennDOT website - www.penndot.pa.gov
Click on Forms, Publications & Maps

APPROVED FOR ISSUANCE BY:

Jonathan R. Fleming
Chief Executive, Highway Administration

Beverly Miller

Beverly Miller, P.E.
Chief Geotechnical Engineer
Bureau of Construction and Materials

Geotechnical Engineering Manual

Publication 293

2022 Edition

pennsylvania
DEPARTMENT OF TRANSPORTATION
www.penndot.pa.gov

PUB 293 (12-22)

[THIS PAGE IS INTENTIONALLY LEFT BLANK]

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
GEOTECHNICAL ENGINEERING MANUAL

TABLE OF CONTENTS

CHAPTER 15 – ROCK SLAKE DURABILITY

[THIS PAGE IS INTENTIONALLY LEFT BLANK]

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
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 roadways 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 using 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 proposed construction 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), 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.

[THIS PAGE IS INTENTIONALLY LEFT BLANK]

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 1 – GEOTECHNICAL WORK PROCESS, TASKS AND REPORTS

CHAPTER 1 – TABLE OF CONTENTS

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 that includes Alternatives Analysis, Preliminary Design, and Final Design. These design phases are completed during Post Transportation Improvement Plan (POST-TIP) activities as specified in 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 most 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 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 enough during the Alternatives Analysis phase. This phase is not needed for projects that are not considering multiple alignments, like routine improvements to existing highways (e.g., including reconstruction, widening, structure replacement, etc.).

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 (e.g., plans, details, specifications, etc.) 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 allows 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 (e.g., segmental, cable stayed, major arches or trusses, steel box girders, movable, etc.)

- Complex bridges (e.g., sharp skewed superstructure, non-conventional piers or abutments, horizontally curved girders, non-conventional piles or caisson foundations, etc.)
- Bridges underlain by complex geology (e.g., 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 as specified in 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 workflow process for geotechnical engineering is graphically shown in Figure 1.1.3-1. This workflow was developed for a major/complex project. The workflow 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 workflow process also shows how structure foundation geotechnical engineering is incorporated into the roadway geotechnical engineering. This workflow 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.

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 provides more accurate budgeting of time and costs.

The 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.

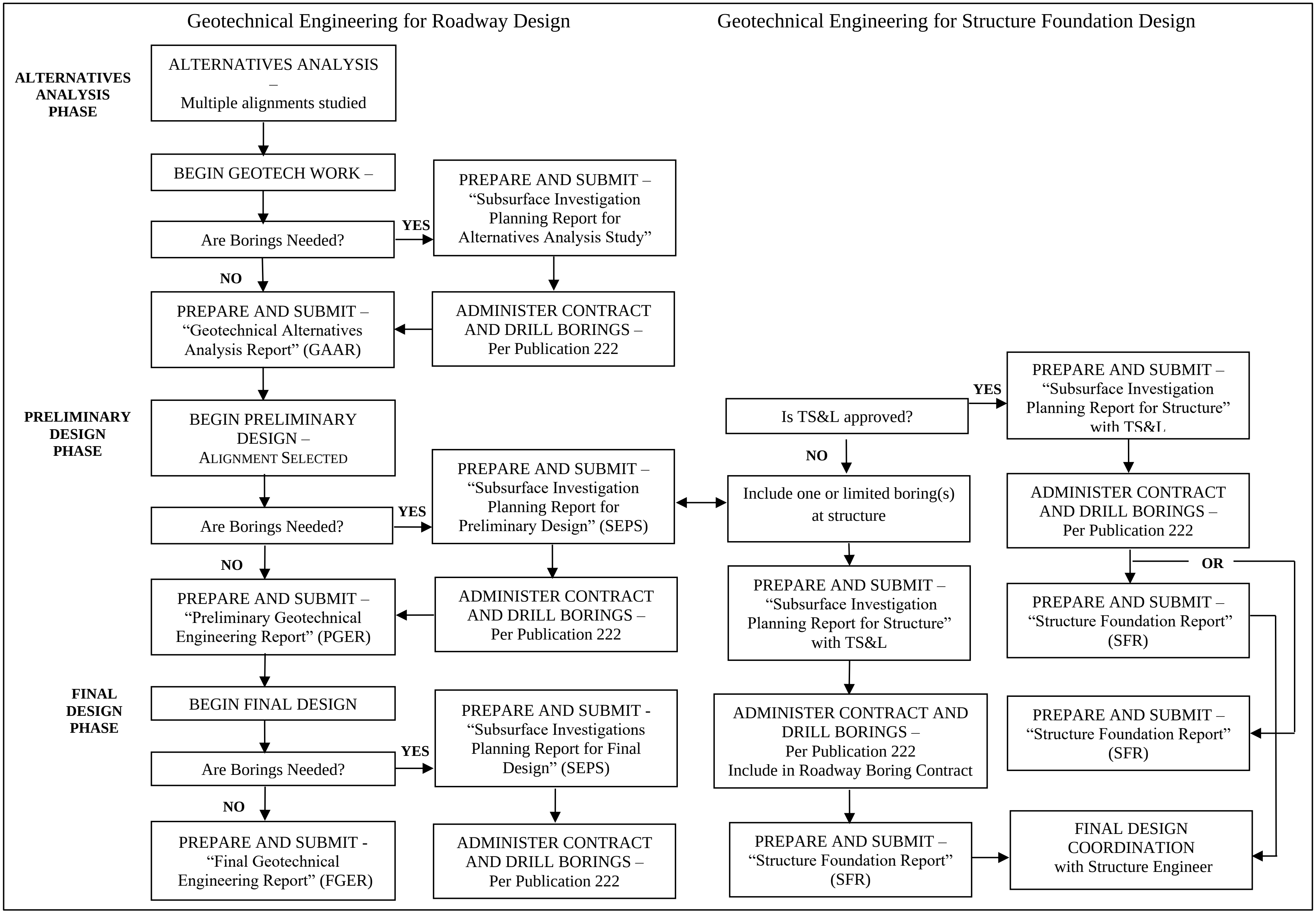


Figure 1.1.3-1 – Geotechnical Engineering Work Process

It is not enough to simply state in the geotechnical scope of work that tasks will be performed according to 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 complete a roadway and/or structure project successfully and thoroughly. 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 (e.g., 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 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 must 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 (e.g., 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) who is a PE or PG licensed in the Commonwealth of PA, and the DGE will attend. It is important that the PPM 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 (e.g., 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.

Before 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. Like 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 (e.g., 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 (e.g., 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 instead 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-producing rock (APR)
- 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 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 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-producing 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 according to other Department 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 (e.g., 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.

Before 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 PPM's and consultants, highway designers, bridge engineers, environmental engineers, and others. When all involved parties have a clear understanding from the beginning of the anticipated, both general and specific, geotechnical activities, it will help to ensure all necessary work is completed within the required timeframe, achieve the desired standard of quality, and 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-Producing 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. Examples include:

- 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 enough 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 (i.e., overexcavation or ground improvement)
- Subgrade stabilization (e.g., overexcavation, lime, cement, etc.)
- Discussion of subgrade for pavement design
- Embankment surcharge, quarantine, wick/sand drains
- Void (i.e., limestone or mining) stabilization
- Non-standard treatments (e.g., extra depth sidehill benching, embankment toe trenches, etc.)
- Anticipated structure foundations
- Anticipated retaining wall types (e.g., 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 the geotechnical information gathered to provide conceptual design recommendations, assess the pros and cons of the various alignments from a geotechnical standpoint. This process provides decision makers with a simple and direct tool for comparing geotechnical alternatives, making it one of the most valuable products of the geotechnical portion of the alternatives assessment. 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. Findings should be presented 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, while keeping the overall project goals and proposed construction in mind. 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 enough 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-producing 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 before 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 significant 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-producing 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 (e.g., solutioning, voids, pinnacles, etc.)
- Identify acid-producing rock
- Determine permissible and anticipated structure foundation types

Again, the subsurface exploration for an alignment must be stopped if, at any time, enough information has been obtained to deem an alignment favorable or unfavorable. This determination will have to be made by the PPM 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-producing 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 (e.g., 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 (e.g., 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 selected from the onset (i.e., 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 must 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:

- Allow 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 projects design and environmental issues because it establishes lateral and longitudinal impact limits. The project subsurface conditions, and ultimately the geotechnical design recommendations (e.g., 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 before 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 but not limited to, 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, and when approvals (e.g., Environmental Clearance, Design Field View, TS&L, etc.) are obtained. **Perform as much geotechnical engineering as possible during the Preliminary Design Phase becuase:**

1. Adequate geotechnical engineering helps 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 (e.g., 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.”

Before starting the geotechnical work, a kickoff meeting must 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 before preparation of deliverables (bullet No. 3), will greatly reduce efforts required for bullet No. 4. At a minimum, the PGM and the

DGE must 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 according to 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 (e.g., 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 Analyses 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-Producing 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 (e.g., 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 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 before 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 enough 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. Enough 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, etc.). 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. Before the completion of the subsurface exploration, the findings must be discussed with the DGE to determine if additional explorations are necessary before 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 according to 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.

If available, environmental documents must be reviewed and discussed with those performing the environmental work for the project 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 enough. See Chapter 4 of this publication for guidelines on laboratory testing.

All proposed laboratory testing must be reviewed and approved by the DGE before 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 (e.g., 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 before 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. Before 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 enough 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 must 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 must 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 (e.g., 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 (e.g., 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 (e.g., 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 most 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

As specified in 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 must 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 must 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 before 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.

Again, it is important to 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 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 enough for final design purposes do not have to be performed. Tasks that were performed during the Preliminary Design Phase, but not to a level enough 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 (e.g., 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.

Before starting the geotechnical work, a kickoff meeting must 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 must 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 according to 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 before 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. Before the completion of the subsurface exploration, the findings must be discussed with the DGE to determine if additional investigations are necessary before 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 instead of the "Subsurface Exploration Planning Submission" outlined in Section 1.5. The subsurface exploration contract and work must be done according to 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.

If available, environmental documents must be reviewed and discussed with those performing the environmental work for the project 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 before 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 ensure 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 must 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 must 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 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 before 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 must review, individually or jointly, the contract documents (e.g., plans, cross-sections, typical details and specifications, etc.) 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 (Department 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 (e.g., 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 proximity of structures with similar non-complex geotechnical conditions. According to 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 before 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 structure borings are proposed (i.e., 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 enough 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 before 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 must 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 (e.g., 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, 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-producing 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 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 must 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/riverbanks and beds, etc.
- Rock types exposed in outcrops, stream/riverbanks and beds, etc.
- Performance of existing facilities (e.g., 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 (e.g., 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 (e.g., restrictions on time of year due to protected aquatic life, filtering drill wash water, Aids to Navigation (ATON), etc.)
- 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 (i.e., 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 (e.g., 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-Producing 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 (e.g., 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 must 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 must note the contractual requirements with respect to the report and the status of the report (e.g., is it a draft or final report, etc.). The report must 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. must be used where possible to present the findings and recommendations clearly and concisely.

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 (e.g., 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

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 allows, 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 (e.g., 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 soil and rock materials 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 (e.g., extra depth sidehill benching, toe trenches, undercuts, etc.)
- Need for settlement treatment (i.e., wick/sand drains, quarantine, or surcharge)
- Pavement subgrade parameter(s) for conceptual pavement design
- Pavement subgrade stabilization (e.g., overexcavation, lime, cement, etc.)
- Permissible soil and rock cut slopes
- Rockfall protection (e.g., catchment zone, fence, etc.)
- Anticipated structure foundation types
- Any anticipated problems or required special treatments for structure or embankment foundations
- Handling/treatment of acid-producing 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 must 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 (e.g., 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-producing rock, etc.).
- Consultation with residents, contractors, and Department personnel that have knowledge of the area.

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

6.3 Reconnaissance - This section must 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/riverbanks and beds, etc.
- Rock types exposed in outcrops, stream/riverbanks and beds, etc.
- Performance of existing facilities (e.g., 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 (e.g., 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 (e.g., 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-Producing 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 must note the contractual requirements with respect to the report and the status of the report (e.g., is it a draft or final report, etc.). The report must 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. must be used where possible to present the findings and recommendations clearly and concisely.

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 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 must 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 (e.g., 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 most 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 (i.e., 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 must 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 must cover an area large enough for someone not familiar with the project to be able to locate it. The project description must 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 must 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 rational 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 as specified in Publication 408, Section 206.

(b) Location and depth of unsuitable soil as specified in 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 (e.g., 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. before 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 (e.g., 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 (e.g., 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-producing rock (APR) 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, etc.)
(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 (e.g., 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 must 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 must 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 must 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 must include:

- Physiographic Setting
- United States Department of Agriculture (USDA) soil types. A map delineating the soil types must 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 must 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 must be divided into two subsections, including Subsurface Conditions and Laboratory Testing Summary.

6.1 Subsurface Conditions – This section must document all subsurface explorations (e.g., borings, geophysics, CPT, test pits, etc.) performed at the site, both during Preliminary and Final Design. Include in this section discussion of instrumentation (e.g., piezometers, inclinometers, etc.) and field tests (e.g., vane shear, pressure meter, etc.) performed. A plan view showing the locations of all borings, CPT's, test pits, geophysical surveys, instruments, etc. must 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 must 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 must 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 must 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 must be considered include:

- Wetlands
- Acid-Producing Rock (APR)
- Hazardous Waste
- Underground Storage Tanks
- Contaminated/Unclean Fill

8.0 Economic Considerations – This section must discuss any economic aspects of the geotechnical work associated with the project. Items that must 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 (i.e., 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 (e.g., streams, rivers, roads, strip mines, etc.).
3. Geologic Map

Other “value added” figures must 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-Producing 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 must 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 must 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 must include standard symbols to identify soil and rock type, and AASHTO classifications and rock type must be indicated adjacent to the boring. The graphical display of the boring must also include SPT N-values, rock core recovery, Rock Quality Designation (RQD), and groundwater readings. Laboratory test results must be shown in a table on the same sheet the boring is displayed. A lab test number must be assigned to each test, and that number must 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 accepted Department software, 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 (Cc, Cr, Cv) 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 must 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 must note the contractual requirements with respect to the report and the status of the report (e.g., is it a draft or final report, are any addendums anticipated, etc.). The Preliminary and Final Geotechnical Engineering Reports must 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. must 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 before 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, before moving into final design. Approval of the parameters by the Department before 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 before 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 (e.g., 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 _____, etc.)
- 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 (e.g., 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 (e.g., 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-producing 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 (e.g., 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 (e.g., 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 (e.g., 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 (e.g., 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 (e.g., type of foundation, footing and pile/shaft tip elevations, etc.) and performance of existing foundations (e.g., 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, redriving, 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 (i.e., 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 (e.g., streams, rivers, roads, strip mines, etc.).
3. Geologic Map

Other "value added" figures must 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-Producing Rock Map, etc.

Appendices

Appendix A – Foundation Plans 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, comprehendible manner. This submission letter from the design consultant to the Department must note the contractual requirements with respect to the report and the status of the report (e.g., is it a draft or final report, are any addendums anticipated, etc.). The report must 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. must be used where possible to present the findings and recommendations clearly and concisely.

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

Before 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, slope stability, etc.). 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 Department 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 (e.g., 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 (e.g., 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 before 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 ensure 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 ensure 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 before installation, to check for obstructions. Such investigations are usually performed with small diameter, rapid penetration drilling equipment without sampling (i.e., are-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, soils 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 ensure 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 ensure 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. Therefore, it is 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 (e.g., 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 ensure 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 ensure 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 ensure the quality of construction, and to ensure 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 deign, 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-Producing Rock (APR)

Acid-producing 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 experience indicates acid-producing rock may be present, should use a qualified geotechnical specialist to observe excavation activities. Identifying acid-producing 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 APR is anticipated, the qualified geotechnical specialist can ensure that the planned mitigation is adequate and properly executed. If it is a project where the potential for encountering APR 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 ensure 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 human-made 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 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 must 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 ensure 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 that includes the DGE 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 (e.g., unstable ground, landslide and settlement prone areas, sinkholes, potential areas of acid-producing 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 –2022
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 2 – RECONNAISSANCE IN GEOTECHNICAL ENGINEERING

CHAPTER 2 – TABLE OF CONTENTS

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 (i.e., soil, rock, and water) at a project site without or before 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/riverbeds/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 allow 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 before 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; therefore, it is 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 enough 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 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 [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 (i.e., 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 website. 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, structure, etc.) 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 an operated or remotely operated aircraft (e.g., plane, helicopter, drone, etc.). If remotely operated 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; therefore, it is the District Geotechnical Engineer's/Project Geotechnical Manager's (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 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 currently housed and made accessible through Pennsylvania Spatial Data Access (PASDA). 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. 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 enough 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, toe bulge, etc.). 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.
- Another source for aerial photography is the Department’s Photogrammetry Asset Management System (PAMS). This site is for a geographic search for available aerial photography as well as traditional survey data (ground control and benchmark information).
- 7.5-minute digital orthophotos covering Pennsylvania with topographic contour overlay can be downloaded from the USGS.

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 on PASDA’s website.

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 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.

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.

2.2.5.2 Pennsylvania Geological Publications

The PGS maintains a web page that includes links to Geologic Publications specific to Pennsylvania. 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. 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 that 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. 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 that contain the location of karst features on a geologic base map. A list of these open file reports can be found on the PGS Open-File Reports on Karst Features. Alternatively, the PGS maintains a 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-producing 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.

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 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 that 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 landsides. Most of these reports and maps focus on the Pittsburgh and surrounding region. These publications are available from the USGS Publications Warehouse. For example, 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. Most of these figures focus on the Pittsburgh and surrounding region. The DCNR stratigraphic column figures are available online as well as the Pittsburgh Geological Society stratigraphic column of the Pittsburgh region.

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 on the PA DCNR Map 11, 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 online.

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.

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 before 1935, and “active” oil/gas wells shown on these maps most likely are no longer active.

Historical mine maps can be obtained from the National Mine Map Repository (NMMR). 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. Federal Emergency Management Agency (FEMA) has Flood Maps searchable by an address or longitude and latitude coordinates. Also, Pennsylvania has a source for Flood Map Information.

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 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) that consists of a large Access database containing data for wells, springs, and groundwater quality throughout Pennsylvania.

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, 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. Currently, 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 before commencement of the subsurface exploration. The PGM should confirm with the DGE as to whether an archeological investigation is required before 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 that could result in early warning signs for possible locations for landslides
- Mapped sinkholes, landslides, mines, spoil piles, etc.
- Areas of gently rolling terrain that are often favorable, but may also be an indicator of subsurface conditions unable to support steep slopes
- Rapidly changing contours and bodies of water that 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 firsthand 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/Riverbeds/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 that 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 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 before 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 according to 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-producing 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; therefore, they 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

Before 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 that 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 riprap around a bridge foundation may indicate a past scour problem, particularly for older structures. Movement of riprap 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 before 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, etc.), 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, etc.). 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-Producing Rock

The presence of acid-producing rock (APR) 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-producing rocks (OFMI Report 05-01.1). This reference should be the first step in determining if a project may be impacted by APR. Check this map and any other publications that may be available to determine if the project lies in an area where APR is known to be present. Also, check if past local projects encountered APR. During field reconnaissance, observe and document any indications of the presence of APR, 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 APR are documented or observed on a project, follow the provisions presented in Chapter 10 for additional information.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 3 – SUBSURFACE EXPLORATIONS

CHAPTER 3 – TABLE OF CONTENTS

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 most 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 (i.e., 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.

Before 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) before performing the exploration. The subsurface exploration must be administered and performed according to requirements 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 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 fully be used 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 (i.e., 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, etc.)
- 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 can meet 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 according to 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, enough 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 enough for small projects.

3.2.2 Subsurface Exploration Planning Meeting

A meeting between the Project Geotechnical Manager (PGM) and DGE should be considered before 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 enough. 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 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 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. Use 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-Producing 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 enough 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 use 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	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. Cuts – Extend borings ten feet below the proposed subgrade elevation. At-grade areas – Extend borings minimum 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.
Bridge	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 according to 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. 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% recovery) or 15 feet of low recovery rock. 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. Drilled shafts bearing/socketed into bedrock – Extend borings a minimum of three shaft diameters below the estimated tip elevation, or two times the maximum shaft group dimension, whichever is greater, but not less than ten feet.
Bridge (cont.)	

Area of Exploration	**Boring Depth[1,2,3]**
	Wingwalls – refer to Retaining Wall requirements.
Culvert	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. Arch culvert or open bottom culvert (i.e., strip footings) – Refer to Bridge (spread footings on rock or soil) requirements.
Retaining Wall	Spread footing on soil – Extend borings a minimum of two times the height of the proposed wall unless rock is encountered. All other retaining wall foundation types – Refer to appropriate Bridge foundation requirements. 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. Soil and rock anchors – Extend borings a minimum of five feet below lowest elevation of anchor.
Sound Barrier Wall Sound Barrier Wall (cont.)	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. 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. Drilled shafts in soil – Extend borings a minimum of four pile diameters, but not less than 10 feet, below estimated tip elevation. 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.
Pavement Design	Extend borings minimum of three feet below the proposed subgrade elevation.

Area of Exploration	Boring Depth[1,2,3]
Dynamic Message Sign	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. 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. 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.
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 stopped in loose cohesionless soil, very soft or soft cohesive soil, fill, or organics. They should also not be stopped in voids, soil/clay seams, or coal seams in bedrock. Borings should be stopped in competent material that is believed to be stable from a settlement and stability perspective. Borings must also never be stopped 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 stopped 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, etc.) 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 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 instead of test borings, but 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 210 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 that 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, etc.), 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 according to 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-feet 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 ensure 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 before widening the 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 not practical or feasible to stake boring locations before 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 arrange use of 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 before performing a subsurface exploration program. A design phase PA One Call must be made by the designer (i.e., Department for in-house projects, or 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. Before 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 before 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. According to 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 according to 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 according to 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 Publication 222, Appendix C 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. According to 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 215 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 used 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 boing 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., Form TR-445A for PennDOT Contract Agreements, or Form TR-446A for Consultant Contract Agreements). 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 (e.g., 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 218 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 of drill rig. Therefore, when possible and if 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 SBSTC", 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 according to 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-feet-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 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. Like hollow-stem augers, sampling and testing can be performed through the casing at desired depths. Casing typically comes in 5-feet-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 that 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; therefore, it is 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 216 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 Spilt-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 according to Publication 222 and ASTM D1586 (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 D1586.

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 that 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 according to Publication 222, Section 216 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 that 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 according to 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, etc.). 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. Before 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 according to 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 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 (i.e., in the same direction it was pushed in the field) 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 snuggly 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, like 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, like 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 Dension sampler is adjustable, it must be manually set before 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 form 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 D2113. Rock core drilling must be performed according to Publication 222, Section 204 and ASTM D2113.

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 according to 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 of 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. According to 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, according to 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 that 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 that 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 human-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. Like rock coring, water is used to cool the bit and clear drill cuttings. Concrete core drilling must be performed according to Publication 222, Section 205 and ASTM C42.

3.6.5 Unsampled 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 enough 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, uses 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 used 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 spilt-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. Publication 222 SBSTC 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 use 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; 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 stopped, or rock core drilling will commence. Whether to stop 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 ensure 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%. 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 according to 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 are typically of poorer quality (i.e., weathered or 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% 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% but greater than or equal to 20%, 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%, additional standard penetration testing may be needed before 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 according to Publication 222, Section 202 is not quite achieved, but little to no sample is recovered after several attempts (i.e., 3 sample intervals or 4.5 feet), consider a short (i.e., 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-footcore 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, etc.) 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 must 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 human-made 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, other mechanical issues, etc.), 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 enough for exploration in horizontal to near horizontal bedded geology and are used on most 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 can complete inclined borings. SPT sampling of the overburden is not feasible in an inclined boring. Boring inclinations up to 30º 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 209 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 instead 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 that 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 according to Publication 222, 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 209 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 before 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 208.

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 before 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.

According to Publication 222, Section 208, water levels must be measured with electronic water level indicators. Care must be taken when using these indicators. Before 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; 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 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 that 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 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. Before 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 before 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.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 4 – LABORATORY TESTING

CHAPTER 4 – TABLE OF CONTENTS

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 like 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; therefore, they 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 instead 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 according to 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 Project Geotechnical Manager (PGM) who must make revisions as appropriate. Note the distinction between the term's **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

Except for 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-Producing 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 allowed 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

Before 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 District Geotechnical Engineer (DGE) before 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 before 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, etc.), 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 enough. 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 enough 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-producing 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 before 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 (e.g., consolidation tests, shear strength and permeability tests, etc.). 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 allow 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 ensure 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 before 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 and free of disturbed material before 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 samples and testing 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 214 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 before 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 before 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 (e.g., 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 before 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 most of the tube. It is not reasonable/appropriate to test a portion of the sample that is unrepresentative of most 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 before performing the testing. Ideally, sufficient samples, and in particular, undisturbed and bulk samples, should be collected during the subsurface exploration to allow “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 are 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 instead 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 (i.e., 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 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 field and laboratory information 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% according to Publication 222, Chapter 3.6.3.1) 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/ft^2. 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 Tests[1] 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
Total material smaller than 0.002 mm			169	1115		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

Test	AASHTO Designation	Equivalent ASTM Designation	Classification AASHTO Sample No.	Total No. Labs	Units	Average	Standard Deviation, S	Coefficient of Variation, CV (%)
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		169	1489		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
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			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

Test	AASHTO Designation	Equivalent ASTM Designation	Classification AASHTO Sample No.	Total No. Labs	Units	Average	Standard Deviation, S	Coefficient of Variation, CV (%)
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			169	516		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
Swell - Percentage of Initial Specimen Height			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			169	514	N/A	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			169	514	N/A	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

Test	AASHTO Designation	Equivalent ASTM Designation	Classification AASHTO Sample No.	Total No. Labs	Units	Average	Standard Deviation, S	Coefficient of Variation, CV (%)
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 according to 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 before 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-Producing 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 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 airtight jar/bag/tube to retain the natural water content.

AMRL Certification Required: Yes

AASHTO Test Method:

T265 – "Standard Method of Test for Laboratory Determination of Moisture Content of Soils".

ASTM Test Method:

D2216 – "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass".

Minimum Required Sample Size (Note one jar ≈ 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 into 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 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:

T88 "Standard Method of Test for Particle Size Analysis of Soils".

ASTM Test Method:

D422 "Standard Test Method for Particle-Size Analysis of Soils".

Minimum Required Sample Size (Note one jar ≈ 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 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:

T88 "Standard Method of Test for Particle Size Analysis of Soils".

ASTM Test Method:

D422 "Standard Test Method for Particle-Size Analysis of Soils".

Minimum Required Sample Size (Note one jar ≈ 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 into 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:

T89 "Standard Method of Test for Determining the Liquid Limit of Soils".

T90 "Standard Method of Test for Determining the Plastic Limit and Plasticity Index of Soils".

ASTM Test Method:

D4318 "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils".

Minimum Required Sample Size (Note one jar ≈ 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 into 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 according to 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:

M145 "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:

D3282 "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.

D2487 "Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)".

Minimum Required Sample Size (Note one jar ≈ 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 airtight 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:

D7263 "Standard Test Methods for Laboratory Determination of Density (Unit Weight) of Soil Specimens".

Minimum Required Sample Size (Note one jar ≈ 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:

T100 "Standard Method of Test for Specific Gravity of Soils" (for maximum particle size of 4.75 mm (No. 4 sieve)).

T85 "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:

D854 "Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer" (for maximum particle size of 4.75 mm (No. 4 sieve)).

C127 "Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate".

Minimum Required Sample Size (Note one jar ≈ 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 into 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 airtight 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:

T267 "Standard Method of Test for Determination of Organic Content in Soils by Loss on Ignition".

T194 "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 T267 must be used.

ASTM Test Method:

D2974 "Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils".

Minimum Required Sample Size (Note one jar ≈ 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 into 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:

T99 "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.
T180 "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:

D698 "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3))". This method is referred to as Standard Compaction.

D1557 "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3))". 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 must 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:

T193 "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 T193), 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:

D1883 "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:

T216 "Standard Method of Test for One-Dimensional Consolidation Properties of Soils".

ASTM Test Method:

D2435 "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. If the layer being modelled is underlain by rock (or other impervious material), it can be appropriate to consider this as a one-way drainage system. Replace the bottom porous stone with an impervious insert to more accurately model field conditions.
5. Extrude Shelby tube in the same direction it was pushed in the field.

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:

T236 "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 ENSURE RESIDUAL SHEAR HAS BEEN OBSERVED.**

ASTM Test Method:

D3080 "Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions".

Minimum Required Sample Size (Note one jar ≈ 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 into 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.
3. Extrude Shelby tube in the same direction it was pushed in the field.

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 (AASHTO T236). IT IS NECESSARY TO RUN THE DIRECT SHEAR TEST WITH SUFFICIENT DISPLACEMENT TO ENSURE 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:

T296 "Standard Method of Test for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression".

ASTM Test Method:

D2850 "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 (e.g., 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 before 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:

T297 "Standard Method of Test for Consolidated, Undrained Triaxial Compression Test on Cohesive Soils".

ASTM Test Method:

D4767 "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 (e.g., 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 consequently rarely performed. Instead 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 (e.g., 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:

T208 "Standard Method of Test for Unconfined Compressive Strength of Cohesive Soil".

ASTM Test Method:

D2166 "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:

D7012 "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 HQ_3 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 that 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:

D5731, “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, 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:

D4644 “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 before 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:

T288 "Standard Method of Test for Determining Minimum Laboratory Soil Resistivity".

Minimum Required Sample Size (Note one jar ≈ 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

AASTHO Test Method:

T289 "Standard Method of Test for Determining pH of Soil for Use in Corrosion Testing".

Minimum Required Sample Size (Note one jar ≈ 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:

T290 "Standard Method of Test for Determining Water-Soluble Sulfate Ion Content in Soil".

ASTM Test Method:

C1580 "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:

T291 "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 D2434 for high permeability soils.

Soil Sample Types: Disturbed (jars and bags) and undisturbed.

AMRL Certification Required: Yes

ASTM Test Method:

D5084 "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:

D1125 "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:

D1293 "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:

D516 "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:

D512 "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 that 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 T99 (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 according to AASHTO T180, which is referred to as the Modified Compaction test.

4.9.2 California Bearing Ratio AASHTO, T193

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.

According to Publication 242, the resilient modulus value for Department projects is estimated through a correlation with the California Bearing Ratio (CBR). The CBR is determined according to laboratory test AASHTO T193-10, 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 T307) cannot be used on Department projects according to 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 (i.e., 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).

According to 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 ¾-inch sieve), obtain an additional sample so that the oversize material can be replaced with finer grained material according to 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 (e.g., sm, sc, gm, gc, etc.) should be tested since the fine-grained portion of these soils often controls the behavior of the soil.

AASHTO T193-10 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 T193-10 allows for testing material in an unsoaked condition, all samples for Department projects must be soaked before 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 before 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/ft^3)
A_m = area of CBR mold (ft^2)

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 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 most 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 according to 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 T216

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 human-made 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 use deep foundations instead 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 properly obtain, preserve, transport, and store 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 T236

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. According to 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 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 allowed by the AASHTO standard. These oversize particles must be removed before 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 allowed 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; therefore, they are allowed to have larger particle sizes.

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 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% of the maximum dry weight density and to within 2% 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 (i.e., 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. Like 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, etc.) or deposition (e.g., varves, etc.).

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 T236 “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 ensure 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 T236 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 T296 and AASHTO T297

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. According to 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. According to 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 allowed 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 allowed by the AASHTO test standard.

4.9.6.1 Triaxial Shear (UU), AASHTO T296

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, or 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 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 T297

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 before 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 allowed 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 the Mohr-Coulomb envelopes, which is like 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 T297

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 before 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 instead 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 and more laboratories are capable of performing.

Some advantages of performing the triaxial shear test instead 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 according to the following equation from a previous ASTM standard (ASTM D2938-86):

$$C = C_a/(0.88 + \left(\frac{0.24b}{h}\right))$$

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 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 before 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 before construction. For example, testing could be performed once during wetter seasons and once during dryer seasons. Water supplies should also be tested shortly (i.e., within 8 weeks) after construction activities are complete 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 to be 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 before 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 telephone 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 enough. Water sampling and testing may not be needed.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 5 – SOIL AND ROCK PARAMETER SELECTION

CHAPTER 5 – TABLE OF CONTENTS

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 assume 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'	ksf				
Undrained (Total) Shear Strength of Cohesive Soils[3]	c, s_u	ksf	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]	
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				

Parameter	Symbol	Unit	Applicable Material	Correlation Estimate	Direct Estimate[1]	Geotechnical Analyses
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 according to Design Manual, Part 3 (DM-3), Chapter 5. Structure Foundation Geotechnical Reports are to include subsurface profiles in the appendices according to 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 before 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
Residual Material formed by disintegration of underlying parent rock or partially indurated material.	Residual sands and fragments of gravel size formed by solution and leaching of cementing material, leaving the more resistant particles; commonly quartz.	Generally favorable foundation conditions
	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.	Variable properties requiring detailed investigation. Deposits generally present favorable foundation conditions.
Alluvial Material transported and deposited by running water.	Floodplain is a deposit laid down by a stream within that portion of the valley subject to inundation by flood waters.	
	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.	Generally favorable foundation conditions; however, detailed investigations are necessary to locate discontinuities. Flow slides may be a problem along riverbanks. Soils are quite pervious.

Geologic Process	Principal Soil Deposits	Pertinent Engineering Characteristics
Alluvial (cont.)	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.	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.
	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.	Relatively uniform in a horizontal direction. Clays are usually subjected to seasonal volume changes.
Glacial Material transported and debris, deposited by glaciers, or by meltwater from the glacier.	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.	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.
Colluvial Material transported and deposited by gravity	Talus is deposits created by gradual accumulation of unsorted rock fragments and debris at base of cliffs.	Previous movement indicates possible future difficulties. Generally unstable foundation conditions.

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; 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-producing 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 (i.e., 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, which 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; 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 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 D1586 (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 according to similar procedures as prescribed in ASTM D1586, 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., according to ASTM D1586). 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 D1586, 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%. 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%, 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 Publication 222, Appendix I, 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 according to Publication 222, Section 202 and ASTM D1586, 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 according to 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, etc.), 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, etc.), direct measurements using field or

laboratory testing (e.g., the internal friction angle from laboratory direct shear testing, etc.) 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 Lab 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, among others, 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 (e.g., poorly graded, well graded, fines content, etc.)
- Shape of particles (e.g., rounded, angular, etc.)
- Unit Weight/Relative Density

The shear strength of fine-grained soils is influenced by:

- Plasticity
- Consistency
- Nature of deposition (e.g., glacial lake, alluvium, colluvium, etc.)
- Stress history of soil deposit (e.g., 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 allow 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; therefore, they are 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 that 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.

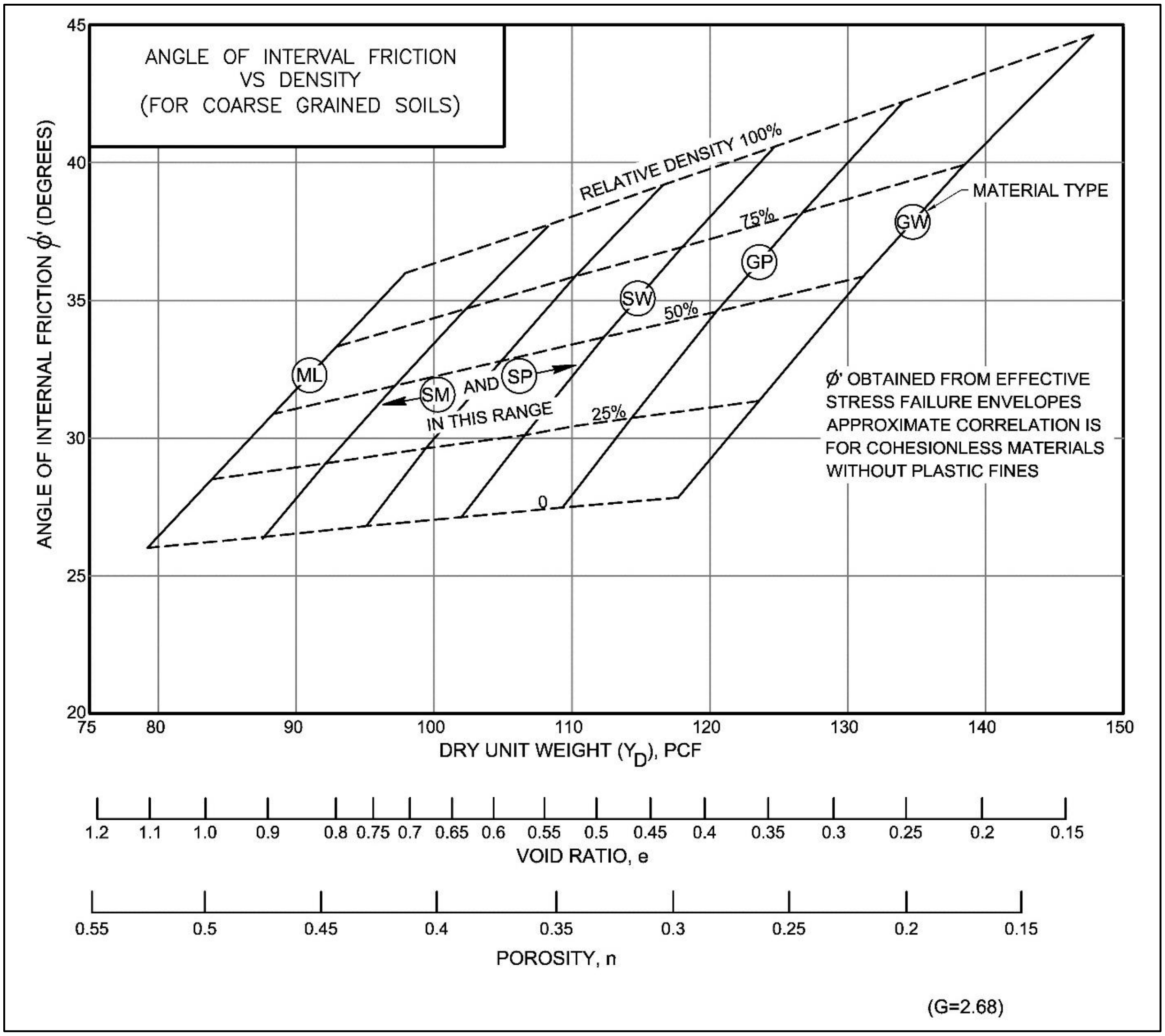


Figure 5.5.3.1.1-1 – Correlations of Effective Angle of Friction with Index Properties for Granular Soils

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 uses 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.

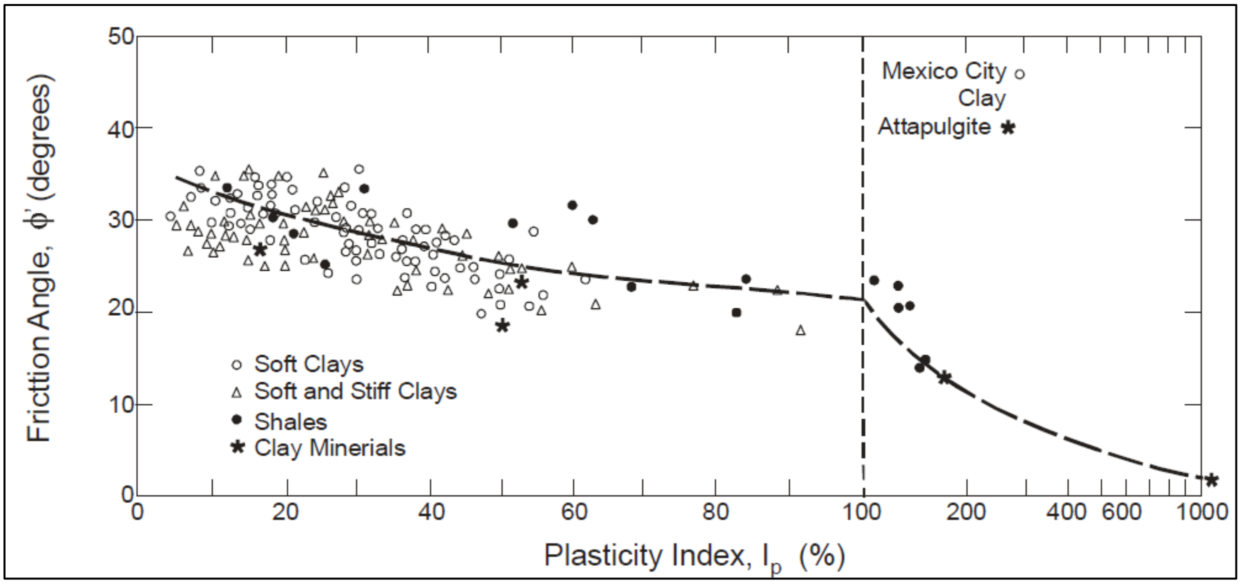


Figure 5.5.3.1.2-1 – Values of Friction Angle, ϕ' for Clays of Various Composition as Reflected in Plasticity Index[1]

Notes: 1. Reference: Terzhaghi, 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 Strength[1]

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 before 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.

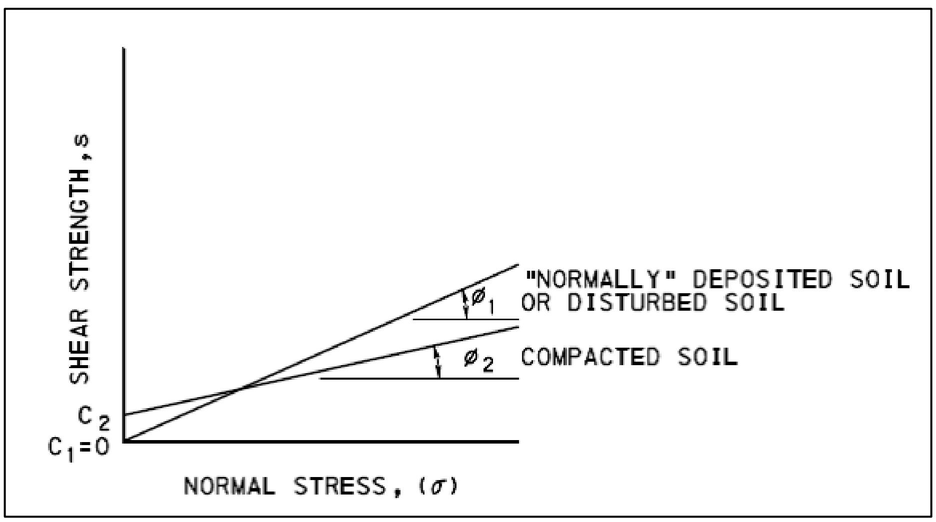


Figure 5.5.3.1.4-1 – Generalized Plots of Shear Strength

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.

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. <u>Soil</u>

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 allowed 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 allow the economical design of embankments, and avoid unnecessary embankment treatments (e.g., 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 according to 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 according to 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 according to ASTM D4644, 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 (lb/ft^3) that includes gravitational acceleration, whereas density is a measure of mass that does not include the effect of gravitational acceleration. Density (Mg/m^3) multiplied by gravitational acceleration of 9.81 m^2/sec will result in a force per unit volume (kN/m^3) that is equivalent to unit weight (lb/ft^3). 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.

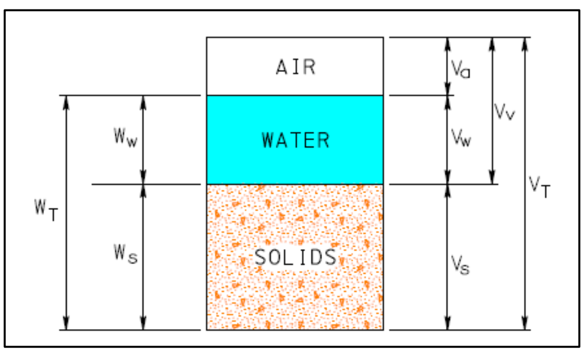


Figure 5.5.3.2-1 – Phase Diagram of Soil

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 State[1]

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, according to 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 indicated in Table 5.5.3.3.2-1.

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 (Terzhaghi, 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 Materials[1]

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: Terzhagi, 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.

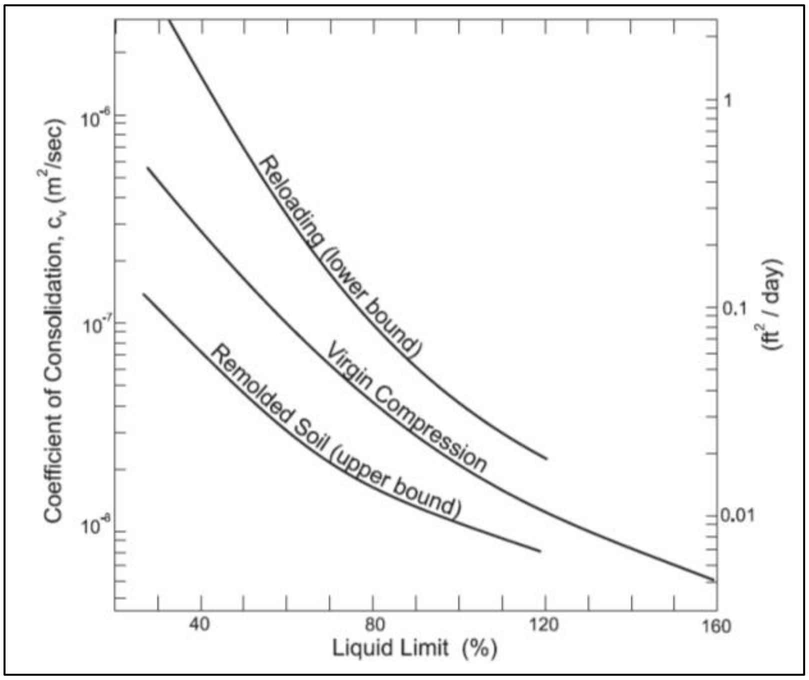


Figure 5.5.3.3.2-1 – Approximate Correlations between c_v and LL[1, 2]

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 before excavation (shrinkage) and the volume of rock fill in an embankment is greater than the volume of rock in-situ before 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 used for soils in Pennsylvania generally range between 10% and 20%. 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% are more typical of soils with a larger fine-grained constituent (A-4, A-6). Shrinkage factors greater than 15% 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 according to project specifications.

Typically, the more massive and harder 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 used for rock formations in Pennsylvania generally range from 0% to 20%. Swell factors for shale may range from 0% to 5% for soft shale, 8% to 10% for medium-hard to hard shale, and up to 10% to 15% for hard silty shale or siltstone. A swell factor of up to 20% 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 T258-81, 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 T258-81, Determining Expansive Soils
- AASHTO T92-97, 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 allowed 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]

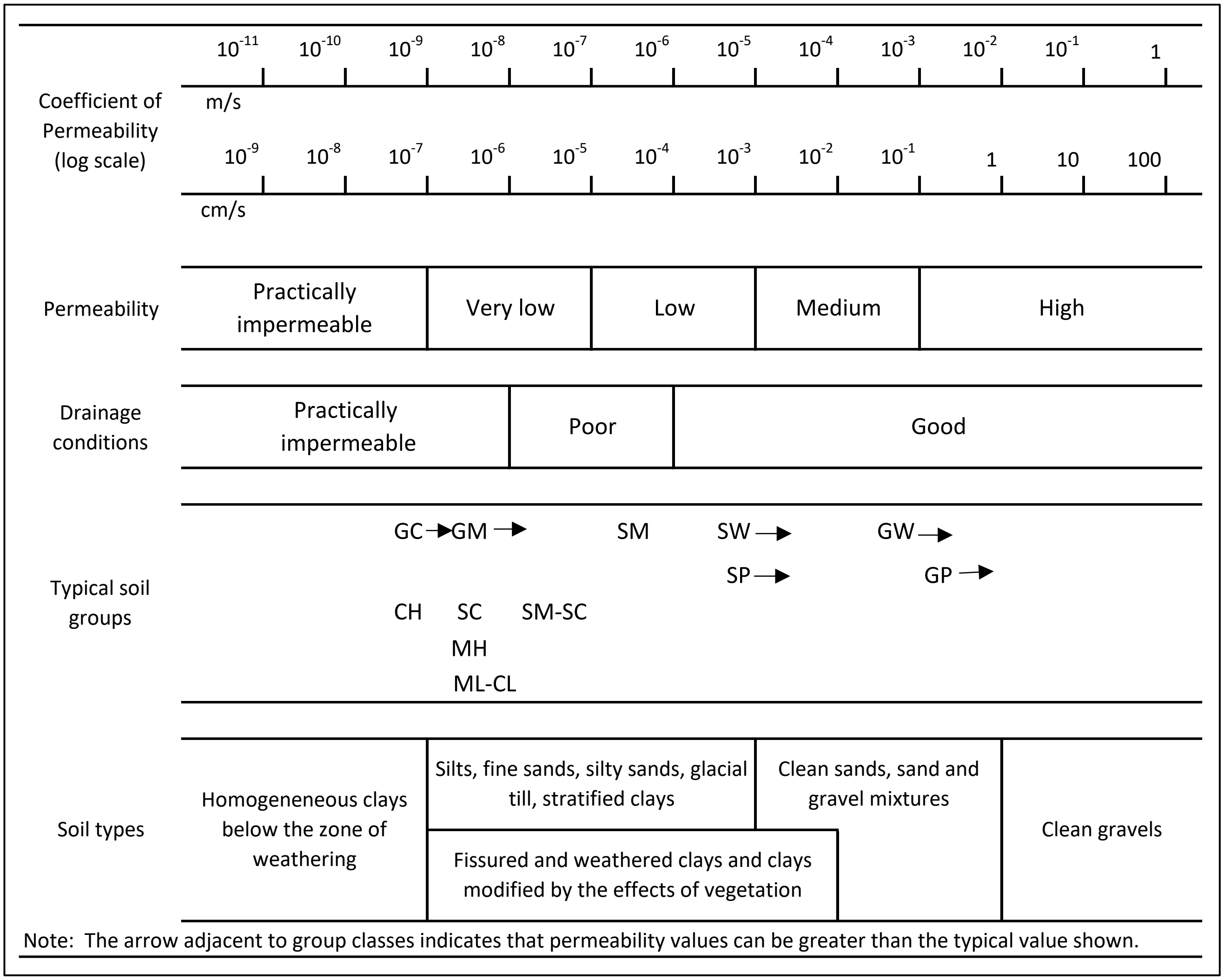
Materials	**Permeability (cm/sec)**
Uniformly graded coarse aggregate	40 - $4x10^{-1}$
Well-graded aggregate without fines	$4x10^{-1}$ - $4x10^{-3}$
Concrete sand, low dust content	$7x10^{-2}$ - $7x10^{-4}$
Concrete sand, high dust content	$7x10^{-4}$ - $7x10^{-6}$
Silty and clayey sands	10^{-5} - 10^{-7}
Compacted silt	$7x10^{-6}$ - $7x10^{-8}$
Compacted clay	Less than 10^{-7}
Bituminous concrete (new pavements)[3]	$4x10^{-3}$ - $4x10^{-6}$
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]



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 enough. 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 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, except for 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 allowed 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 into the calculation of RQD) and joint condition are important. **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.

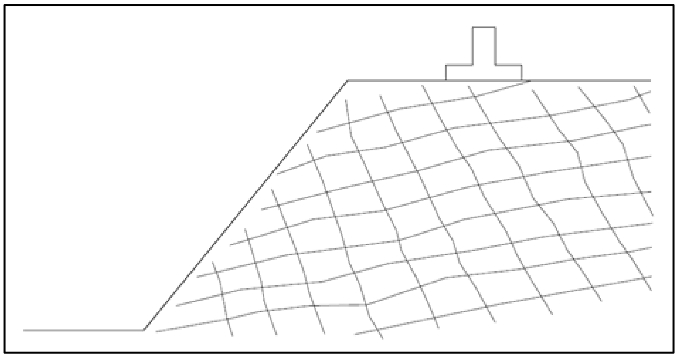


Figure 5.5.4-1 – Spread Footing on Rock above a Rock Cut

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 according to 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 according to 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 according to 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 should be investigated when foundations are subjected to very large loads or where the settlement tolerance may be small. Reference Publication 15 (Design Manual Part 4 (DM-4), Part B, Chapter 10, Section 10.4.6.5) to estimate the elastic modulus of intact rock (E_R). For spread footings on rock, the elastic modulus of rock mass (E_m) is the lesser modulus as determined by DM-4 equation 10.4.6.5-1 or

10.4.6.5-2. Note that RMR is used in the determination of the elastic modulus of the rock mass for spread footings. For all other applications, the elastic modulus of the rock mass can be estimated based on the determined value or range of values for GSI and the elastic modulus of intact rock (E_R) according to AASHTO Table 10.4 6.5-1. AASHTO Table C.10.4.6.5-1 contains ranges of E_R for various rock types. 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 GSI[1]

Expression[2, 3, 4, 5]	**Remarks**	**Reference**
$E_m = \left(q_u\left(\frac{6.89}{100}\right)\right)^{0.5}\left(10^{\left(3+\frac{(GSI-10)}{40}\right)}\right)$ $for\ q_u \leq 14.5\ ksi$	Accounts for rocks with q_u < 14.5 ksi	Hoek and Brown (1997), Hoek et al. (2002)
$E_m = 145.14\ \left(10^{\left(\frac{GSI-10}{40}\right)}\right)$ $for\ q_u > 14.5\ ksi$		
$E_m = \left(\frac{E_R}{100}\right)e^{GSI/21.7}$	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. 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. According to 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, among other data 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, etc.) 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, etc.) do not agree. The reasons for the differences must 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 to reduce design costs, these are generally not realistic points of view. Consider the construction cost savings (e.g., 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 that may also reduce the footing size. Also, consider the construction cost savings (e.g., 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.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 – 2022
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 6 – SOIL SLOPE STABILITY

CHAPTER 6 – TABLE OF CONTENTS

6.1 INTRODUCTION

This chapter of the publication provides guidelines, recommendations, and considerations for soil slope stability in regard to the design and construction of new soil slopes, the rehabilitation of existing soil slopes, and guidelines to model and remediate slope failures. The information in this chapter has been formulated based on review and analysis of the latest Department practices and available published literature and software.

The presented information covers a range of topics including influences and concerns with excess pore water pressure, surficial failure surfaces, surcharge loads, and occasions when analysis is required to assess the short- or long-term stability of a slope. Analysis involves a combination of field observations, interpretation of subsurface investigations, laboratory testing, and instrumentation, culminating in development of a slope model for use in computer analyses.

The use of back analysis is discussed to verify the validity or accuracy of slope stability models. The guidelines and considerations presented in this chapter should be used during project reconnaissance, evaluation/design, and maintenance of soil slopes. Rock slope stability is discussed in a Chapter 8 of this publication.

6.1.1 Purpose

The purpose of this chapter is to determine whether a proposed slope meets required safety and performance criteria, as specified later in this chapter or indicated for project specific requirements. Slope stability analyses are used to determine stability conditions of existing natural or constructed slopes and evaluate the influence of proposed remediation methods, if required. The stability of slopes must be considered during the design of roadway cut and fill slopes, embankment widenings, and structure foundations. Slope stability analysis is also critical for assessing and mitigating slope failures and for potential seismic activity.

6.1.2 Special Cases and Requirements

Like most geotechnical related issues and due to the numerous variables associated with projects, it is not possible to provide discrete slope stability requirements that will satisfy all projects. Although the slope stability requirements provided herein are appropriate for most Department projects, these requirements may not satisfy the needs of projects having atypical conditions or needs.

If project specific slope stability requirements that differ from those presented in this publication are needed, they must be specified in the geotechnical scope of work. Any proposed changes to the slope stability requirements provided in this publication must be approved by the Chief Geotechnical Engineer (CGE). To obtain approval, thorough documentation must be submitted, including supporting technical justification and accompanying analyses that the proposed changes are both required and appropriate for the specific site conditions and circumstances.

6.2 SUBSURFACE INFORMATION

In order to evaluate an existing or proposed slope and perform meaningful slope stability analyses, the subsurface conditions must be adequately understood. The accuracy of the results from the slope stability analyses are directly dependent upon the parameters and boundaries input by the user into the computer software. Adequate and reliable input data obtained from subsurface exploration, laboratory testing, and instrumentation programs (as appropriate) will yield the most meaningful and realistic results. Assumptions that are used for input into the slope stability analyses will reduce the reliability of the results.

6.2.1 Minimum Subsurface Information Requirements

When assessing slope stability, the following subsurface information is required at a **minimum**:

- Soil and rock (where applicable) strata and orientation/slope of strata
- In-situ soil testing (SPT and/or CPT) for applicable soil layers
- Groundwater level(s)
- Laboratory classification test results, including natural moisture content, sieve and hydrometer analyses, and Atterberg limits
- Laboratory in situ shear strength testing: Preferred for non- critical situations, mandatory as specified below

6.2.2 Preferred Subsurface Information Requirements

It is a best practice that lab testing for shear strength parameters be performed whenever practical. For high risk, high value, and critical situations and conditions, laboratory testing of representative project specific materials to determine shear strength parameters is mandatory.

A thorough review and evaluation of the soil strata is critical so that a representative soil profile can be modeled. It is extremely important to define any thin, weak layers present (e.g., silt-clay varves, collapsible soils, etc.) as they could control the stability of the slope. In some cases, additional information may be required to construct an adequate model, including:

- Laboratory and/or in-situ shear strength testing
- Fluctuations in groundwater level obtained from automated piezometers (i.e., vibrating wire with data logger)
- For the case of an existing landslide, inclinometer information that identifies the location of the failure plane and magnitude of movement.

The location and characteristics of the critical soil strata are usually obtained by taking continuous split spoon or thin-walled tube samples. Borings should be progressed in sufficient quantities and to sufficient depth to adequately delineate and define the lateral extent and depth of the relevant subsurface materials. In a potential slide area, borings should be taken at the top and bottom of existing or proposed slopes, if possible. For very large or complex conditions, consideration should be given to mid-slope borings, if practical. The number of borings required longitudinally depends on the continuity of the soil conditions and the extent of the possible problems as discussed in Chapter 3 of this Publication. The required depth of borings is dictated by the potential size of the unstable zone, and the nature of the materials encountered.

Knowledge of the past geologic conditions at a site, including past performance, will aid in the determination of ground conditions and the development of a realistic geotechnical model. Also, a representative slope geometry is crucial for slope stability analysis, and should include applicable locations of streams, roads, structures, and utilities. Surfaces needed to reflect slope geometry can be derived from photogrammetric mapping, field surveys, or published topographic maps as discussed in Chapter 2 of this Publication.

It should be noted that a representative slope geometry does not have to be highly accurate. Even when the slope geometry is obtained by means that provide a high level of precision, it may not be necessary to reflect every element in the geometric model. For example, while the presence of a swale may be critical to understanding the dynamics driving the stability of a slope, its representation in the geometry of the slope may have minimal impact. Simple, but representative, is the best rule for developing a slope stability model. An overly complex model usually adds little to the final analysis other than more time for model preparation and more opportunity for error. A notable exception is when thin weak layers are present as these may be the driving factor of potential instability and, therefore, critical to a representative model.

As discussed later in Section 6.4, the type, quality, and the confidence in the subsurface information will have an impact on the required slope stability safety factor (FS). When the appropriate subsurface information is not obtained, design assumptions (e.g., soil strength, groundwater level, etc.) must be made. These assumptions can lead to overly conservative designs that may include costly construction details (e.g., flattened slopes, slope treatments, right-of-way acquisition, select embankment materials, rock toe trenches, retaining structures, etc.) that may not be necessary.

Conversely, assumptions in subsurface conditions and soil parameters can also lead to inadequate designs, with actual safety factors lower than indicated by results of analyses. In such circumstances, slope failures during or after construction may occur, or it is possible that more frequent maintenance of the slope may be necessary, both of which are costly. Therefore, the relatively minimal time and cost associated with obtaining complete and good quality subsurface information is insignificant compared to costs incurred during and/or after construction as the result of inadequate information.

Once the subsurface information is obtained and the laboratory testing is performed, parameters for the soil strata must be selected. Parameters necessary for slope stability analyses include moist and saturated unit weight, and shear strength parameters (i.e., internal friction angle and/or cohesion). A detailed discussion on soil parameter selection is included in Chapter 5 of this publication.

6.3 METHODS OF STABILITY ANALYSIS

Limit equilibrium methods must be used to analyze slope stability. All limit equilibrium methods use the concept of static equilibrium to calculate the slope stability FS. Methods include both force and moment equilibrium. The slope stability FS is calculated by dividing the total resisting force or moment by the total driving force or moment. A factor of safety (FS) greater than 1.0 indicates a stable slope (i.e., total resisting forces or moments are greater than total driving forces or moments), whereas a FS less than 1 indicates an unstable slope (i.e., total resisting forces or moments are less than total driving forces or moments).

Limit equilibrium methods are two-dimensional and use one of two basic approaches to model slope stability. One approach is to calculate equilibrium for the sliding mass as a whole. The infinite slope method and the slip circle method use this single-free-body approach. The other approach is to divide the sliding mass into vertical sections/slices and calculate equilibrium for individual slices. This approach is referred to as the method of slices. The method of slices is used in numerous procedures including Ordinary Method of Slices, Simplified Bishop Method, Spencer's Method, Janbu, Morgenstern-Price, and others. Analysis discussed in this section will focus on the method of slices.

6.3.1 Method of Slices

Limit equilibrium approaches using the method of slices must be used for slope stability analyses for Department projects. These analyses are performed by assuming a failure surface and calculating the FS associated with the assumed failure surface. Numerous failure surfaces must be analyzed, and the failure surface with the lowest FS will typically identify the critical failure surface. However, there are some situations where the lowest FS may not correspond to the critical failure surface, such as, for example, as with some surficial/shallow failure surfaces. Engineering judgment is required to evaluate the location of the failure surfaces and the corresponding safety factors to determine which is most critical.

6.3.2 Computer Software

Computer software is ideal for method of slices analyses because once the model is entered into the program, safety factors can be calculated for hundreds or thousands of failure surfaces in a few seconds. By hand, such calculations are extremely tedious requiring two to four hours per failure surface with the aid of spreadsheet software such as Excel. Understanding the mathematical process being performed provides an understanding of the power of slope stability computer programs. However, in order to effectively harness the power of slope stability computer programs, an understanding of the theory and mathematical processes being performed is critical.

It is necessary to analyze numerous surfaces to ensure that the most critical failure surface is identified. Any validated and licensed software can be used on Department projects to preliminarily analyze slope stability. **However, final slope stability analyses must be performed and presented to the Department using only accepted Department software.** For a list of currently accepted Department software for slope stability analyses, visit the following link for Accepted Commercially Available or Consultant Developed Software.

Multiple limit equilibrium methods are offered by the accepted Department software, but, for consistency purposes, only two methods are allowed to be used for Department projects. These include the Simplified Bishop Method and the Simplified Janbu Method. The use of alternate methods must be approved by the CGE. To obtain approval, thorough documentation must be submitted including supporting technical justification and analysis to establish that the method is both required and appropriate for the specific site conditions and circumstances.

6.3.2.1 Simplified Bishop Method

The Simplified Bishop Method, which is also referred to as the Modified Bishop Method, must be used when analyzing circular failure surfaces. This method assumes no shear force between the slices. The Simplified Bishop Method satisfies moment equilibrium about the center of the failure circle for each slice, and vertical force equilibrium for each slice. The Simplified Bishop Method must be used for any analysis considering circular failure surfaces (i.e., the radius of the failure surface remains constant). To verify the Simplified Bishop Method, it is recommended to use the Spencer Method.

6.3.2.2 Simplified Janbu Method

The Simplified Janbu Method must be used when analyzing non-circular failure surfaces (i.e., the radius of the failure surface is not constant), and for wedge failures. Like the Simplified Bishop Method, the Simplified Janbu Method assumes there is no shear force between slices, and it satisfies vertical force equilibrium for each slice. However, instead of satisfying moment equilibrium, the Simplified Janbu Method satisfies horizontal force equilibrium for each slice. Also, this method introduces correction factors to increase the FS comparable to other methods.

6.3.3 Alternative Method

For extremely complex slope stability issues, limit equilibrium methods may not adequately model the anticipated stability failure mode(s). Additionally, limit equilibrium methods do not provide a quantitative estimate of movement, which for complex situations may be needed. In these cases, more rigorous methods of analysis, like a finite element analysis, may be used. If a more rigorous method is used, limit equilibrium methods must be used in conjunction with the requirements of this publication. The results of both stability methods must be compared, and if they differ considerably, justification must be provided as to which analysis method is deemed more appropriate.

When the need for more rigorous methods is known or anticipated during the scoping phase of a project, include these additional slope stability analyses requirements in the geotechnical scope of work. If the need for these analyses is identified during the project, the geotechnical scope of work and price may have to be amended. Thorough documentation must be submitted, including supporting technical justification and analysis to establish that the proposed more rigorous method is both required and appropriate for the specific site conditions and circumstances.

6.4 FACTOR OF SAFETY FOR SLOPE STABILITY

The need to perform computer slope stability analyses to estimate the required FS for slope stability is dependent upon several components, including:

- the height of the slope
- the slope ratio/angle
- the quality and variability/complexity of the subsurface information
- the performance of the existing slope (i.e., for widenings)
- the location of groundwater
- whether a structure is within or above the slope
- permanent versus temporary condition
- static or seismic conditions

Table 6.4-1 provides a detailed summary of the slope stability analysis and FS requirements that are described in greater detail in the following subsections. Figure 6.4-1 displays a flow chart to assist in determining the required minimum FS requirements for slope stability for various conditions. The table and flowchart offer the same information and FS requirements.

Although all slopes must be evaluated with respect to slope stability, computer analyses to estimate the FS for slope stability are not always required. Regardless, it is required and necessary to obtain the minimum subsurface information outlined in Section 6.2 to understand the subsurface conditions and evaluate the slope.

Table 6.4-1 – Analysis and Factor of Safety (FS) Requirements

Construction Condition	**Slope Requirements (Applicable Section)**	**Subsurface Information Required (Applicable Section)**	**Minimum FS Required**
Temporary or Permanent Embankment	**All of the following:** Slope < 20 ft high; slope is 2H:1V or flatter; embankment meets Pub 408, Sec 206; estimated $\phi \geq$ 30 degrees; grade below slope is 4H:1V or flatter; no structure within or above slope. (Section 6.4.1.1)	Minimum (Section 6.2.1)	Computer Analysis and Minimum FS at Department Discretion
Embankment Widening	**All of the following:** Section 6.4.1.1 requirements, as well as existing slope is 1.5H:1V or flatter; existing embankment is benched before fill construction; estimated ϕ of existing embankment $\geq$ 30 degrees; existing embankment shows no signs of instability. (Section 6.4.1.1)	Minimum (Section 6.2.1)	
Cuts	**All of the following:** Slope < 20 feet high; slope 2H:1V or flatter; Estimated $\phi \geq$ 30 degrees; slope above cut 10H:1V or flatter; slope below cut 4H:1V or flatter; groundwater level at or below toe of cut; no structure within or above slope. (Section 6.4.1.2)	Minimum (Section 6.2.1)	Computer Analysis and Minimum FS at Department Discretion
Existing Cut Widening	**All of the following:** Section 6.4.1.2 requirements, as well as existing cut slope is as steep or steeper than proposed cut; existing cut shows no signs of instability. (Section 6.4.1.2)	Minimum (Section 6.2.1)	
Permanent Embankment	Slope < 40 feet high (Section 6.4.2)	Minimum (Section 6.2.1)	1.5
	Slope < 40 feet high (Section 6.4.2)	Preferred (Section 6.2.2) and Select Model (Section 6.4.2)	1.3
	Slope $\geq$ 40 feet high (Section 6.4.2)		1.3
Slope Located Above Structure - Shallow Foundation	Slope < 20 feet high (Section 6.4.3.1)	Minimum (Section 6.2.1)	1.5
	Slope < 20 feet high (Section 6.4.3.1)	Preferred (Section 6.2.2) and Select Model (Section 6.4.2)	1.3
	Slope $\geq$ 20 feet high (Section 6.4.3.1)		1.5
Slope Located Below Structure - Shallow Foundation	Slope of any height whose stability could impact the shallow foundation of the structure. (Section 6.4.3.1)	Preferred (Section 6.2.2) and Select Model (Section 6.4.2)	1.5
Slope with Structures - Deep Foundation	Follow Sections 6.4.1 and 6.4.2 as appropriate; do not model structural elements of deep foundations into stability analysis; structural elements may not be considered as contributing to slope stability unless specifically designed with additional capacity for that purpose (Section 6.4.3.2)		

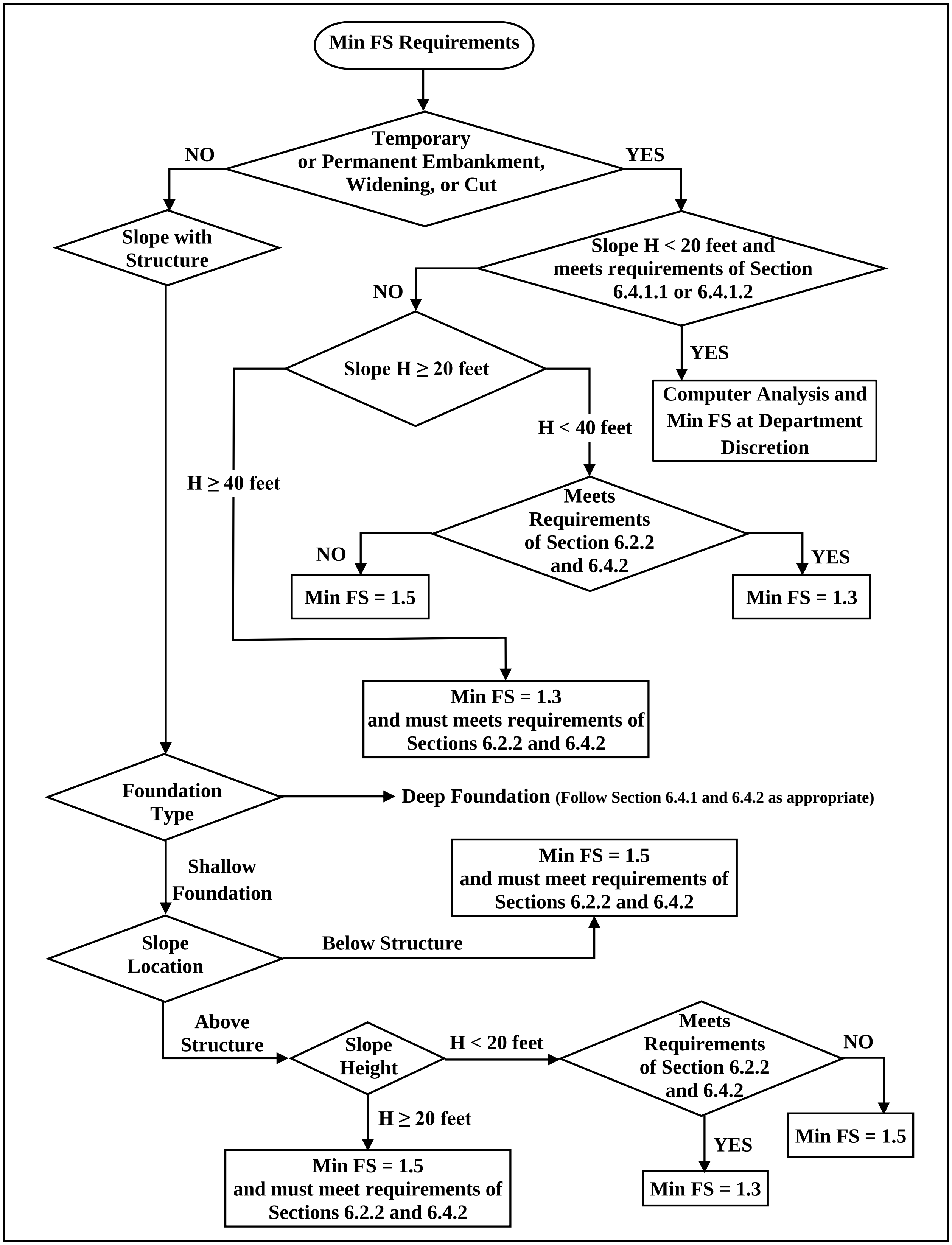


Figure 6.4-1 – Minimum Factor of Safety Requirements for Slope Stability

6.4.1 Slopes That May Not Require Computer Analysis

Computer analyses to estimate the slope stability FS **may** not be necessary for certain situations for soil or rock embankment and cut slopes or widenings. An assessment of the site conditions in context with all subsurface information and all other relevant project data will be required to decide whether computational slope stability analyses is required. If after determining site conditions are favorable and/or a low risk situation, and subsurface conditions, all other relevant conditions, and the requirements in the following subsections are all satisfied, the District Geotechnical Engineer (DGE) may opt to forego computational slope stability analyses.

6.4.1.1 Proposed Embankments and Embankment Widenings

Unless otherwise indicated by the Department, computer slope stability analyses **may** not be necessary for embankment slopes meeting **all** the following requirements:

- The minimum subsurface information discussed in Section 6.2.1 was obtained
- Proposed embankment is less than 20 feet in height, measured vertically from toe to top of proposed embankment
- Proposed slope is 2H:1V or flatter
- Embankment will be constructed as specified in Publication 408, Section 206
- Estimated friction angle of foundation material (to a depth of at least the height of the embankment) is at least 30 degrees
- Ground slope beneath the embankment and beyond the toe of the embankment for a distance of at least 20 feet is 4H:1V or flatter
- A structure is not proposed or currently within or above the slope
- For slope widening, in addition to above, the following requirements apply:
    - The existing slope is l.5H:1V or flatter;
    - The existing embankment will be benched before fill placement;
    - The estimated friction angle of the existing embankment is at least 30 degrees;
    - The existing embankment shows no indication of instability.

6.4.1.2 Proposed Soil Cuts and Soil Cut Widenings

Unless indicated otherwise by the Department, computer slope stability analyses **may** not be necessary for soil cut slopes meeting **all** the following requirements:

- The minimum subsurface information discussed in Section 6.2.1 was obtained
- Proposed cut is less than 20 feet in height, measured vertically from toe to top of proposed cut
- Proposed slope is 2H:1V or flatter
- Estimated friction angle of soil (from top of cut to a depth of at least the height of the cut below the toe of cut) is at least 30 degrees.
- Ground slope above the top of cut is 10H:1V or flatter, and ground slope beyond the toe of the cut slope is less than 4H:1V.
- Groundwater level is at or below the toe of the proposed cut
- A structure is not proposed or currently within or above the slope

- For cut slope widening, the existing cut slope is as steep or steeper than the proposed cut and shows no indication of instability.

6.4.2 Permanent Slopes

Computer slope stability analyses are required for soil embankment and cut slopes not meeting the conditions described in Section 6.4.1, and/or as otherwise required by the Department. The minimum required FS for slope stability for the final condition (i.e., long-term drained soil strength) for slopes less than 40 feet in height is 1.5 when only the minimum required subsurface information discussed in Section 6.2.1 is used for the analyses. If preferred subsurface information is obtained and used for the slope stability analyses, a FS of 1.3 is allowed, unless otherwise directed by the Department. Additionally, for slopes greater than or equal to 40 feet in height, a select slope stability model is required for analysis and the FS for the analysis must meet or exceed a FS of 1.3, unless otherwise directed by the Department. Select slope stability models must include the following:

- The minimum subsurface information requirements discussed in Section 6.2.1.
- Development of a design model based on the subsurface information. The design model must represent subsurface conditions and existing soil strata with a high level of confidence. Obtain subsurface information beyond that required in Section 6.2.1 if required to adequately delineate subsurface layers and subsurface conditions (including water levels), especially when dealing with sensitive, high risk or complex subsurface conditions.
- Laboratory shear strength testing of the soil layer(s) that influence the stability FS. That is any soil layer that:
    - By contrast has significantly different shear strength
    - Is enveloped (even marginally) within the failure or potential failure surface, especially materials on the resisting side of circular failure surfaces, or providing significant resistance along planar the failure surfaces
- High quality shear strength tests are conducted on each layer to provide a high level of confidence in the shear strength value(s) used in the analysis model. A high-quality shear strength test would consist of four test points resulting in a nearly straight line when plotted.

If after obtaining the required subsurface information, performing the necessary laboratory testing, and variation in the information/data does not allow development of a design model with a high level of confidence, sensitivity analyses must be performed to estimate the possible variation in the slope stability FS. A minimum FS of 1.3 must be achieved for the model developed from the conservative information and data. When less conservative or average information and data is used, or if the slope contains, provides support to, or impacts a structural element, a minimum FS of 1.5 must be achieved for the model.

6.4.3 Slopes with Structures

For slope stability analysis, an adequate design is evaluated based upon achieving a satisfactory FS. A slope stability FS is a function of both loads and resistance. The FS is calculated as the ratio of resisting moment or load, over driving moment or load. It is comparable to Allowable Stress Design (ASD) for structures.

In Load and Resistance Factor Design (LRFD), the concept of a FS does not apply because the equivalent of partial factors of safety are applied incrementally for structural analysis to both loads and available resistance. Loads are typically increased with a load factor, and available resistance is decreased by resistance factors. Load factors are set based upon the specific conditions of a design load case, and resistance factors are assigned based upon reliability of strength properties of the structural components. The "factor of safety" concept used for slope stability does not fit well with LRFD for structures.

To address this problem, AASHTO Section 11.6.2.3 – Overall Stability, of the LRFD Bridge Design Specifications, provides guidance for retaining structures, shallow foundations within or adjacent to a soil slope, or structures impacted by soil slopes when determining the required overall slope stability FS. This not only includes any slope that provides support to a shallow foundation or retaining structure, but also for any critical failure surfaces passing within material providing support to (i.e., passing within the stress envelope of) these structures. These requirements also apply for any structure or substructure similarly impacted by a soil slope.

Rather than directly indicating the required FS for slopes involving or impacting structures, AASHTO LRFD treats the required slope stability FS as if it were a resistance factor. AASHTO LRFD Bridge Design Specifications, Section 11.6.2.3 requires soil slopes be evaluated at the Service I Load Combination using a resistance factor of $\phi = 0.65$ where the slope contains, supports or impacts a structural element, and when the geotechnical parameters are based on limited information. Taking the reciprocal of the resistance factor ($1/\phi$) yields a 1.5 required FS for slopes involving structures. **(Note: the resistance factor ϕ, should not be confused with the angle of internal friction of soil, ϕ. The two values just share common symbology.)** The FS requirements for slopes involving structures are further defined in the following paragraph.

6.4.3.1 Structures with Shallow Foundations

Slope stability analysis is required for structures supported on shallow foundations regardless of the height of the slope. Design requirements will change depending on where the structure is in relation to the slope. If the slope is located above the structure with a shallow foundation, there are three slope requirements that determine the minimum FS. For slopes less than 20 feet in height (measured from the toe to top of slope), the minimum required FS is 1.5 when only the minimum required subsurface information discussed in Section 6.2 is used, and is 1.3 when preferred subsurface information discussed in Section 6.4.2 is used. When shallow foundations are proposed within or near a soil slope greater than or equal to 20 feet in height, preferred subsurface information as defined in Sections 6.2.2 and 6.4.2 must be used, and the minimum required slope stability FS is 1.5. Slopes located below the structure that could influence the stability of the shallow foundation have a FS of 1.5 regardless of the height of the slope. Preferred subsurface information must be used when the slope is below the structure.

6.4.3.2 Structures with Deep Foundations

Embankments with structures supported on deep foundations may be designed according to the criteria for embankments without structures, as referenced in Section 6.4.1 or Section 6.4.2 as appropriate, if the deep foundations extend to a firm layer below any possible failure plane. Note that while the presence of deep foundations most likely will improve the stability of the embankment due to the rigid inclusions, these elements are not to be modeled in the slope stability analysis; additional capacity must be built into structural foundation elements specifically for the purpose of improving slope stability.

6.4.4 Temporary Slope Conditions

Slope stability analysis for a temporary construction condition may be required for a project. Temporary conditions that warrant slope stability analyses include a variety of circumstances including a temporary soil cut required to build a structure or install a utility, or a temporary widened/steepened embankment for maintenance and protection of traffic. When these conditions are proposed in the construction documents, it is the designer's responsibility to perform stability analysis. If the temporary conditions are a result of means and methods selected by the contractor, then it is the contractor's responsibility to perform the necessary slope stability analysis. For slopes less than 20 feet in height, that do not have a structure within or on top of the slope, and when preferred subsurface information is used, a minimum slope stability FS of 1.2 is acceptable for temporary conditions with approval from the DGE.

The above temporary short term construction conditions include construction situations involving a period of days as opposed to weeks or months. Longer term temporary construction conditions must meet the long-term slope stability requirements specified in this chapter or as required in DM-4 when structures are involved.

6.4.5 Rapid Drawdown

A rapid drawdown condition arises when a saturated slope is subjected to a sudden or rapid reduction of the external water level. Extended high water flow levels followed by a relatively rapid drop in flow elevation leaves the low permeability, poorly draining soil, saturated. The reduced external ground water level (drop in stream water level elevation) can result in either a reduction of external hydrostatic pressure, thereby reducing an external stabilizing force, or as a change in the internal pore water pressures. These impacts are illustrated in Figure 6.4.5-1.

For analysis purposes, it is assumed that the change from condition "a" and condition "b" in Figure 6.4.5-1 is instantaneous. Therefore, this allows modeling condition "b" in a slope stability computer analysis. Required factors of safety for a rapid drawdown condition would be as required in Sections 6.4.2 through 6.4.4 in this chapter.

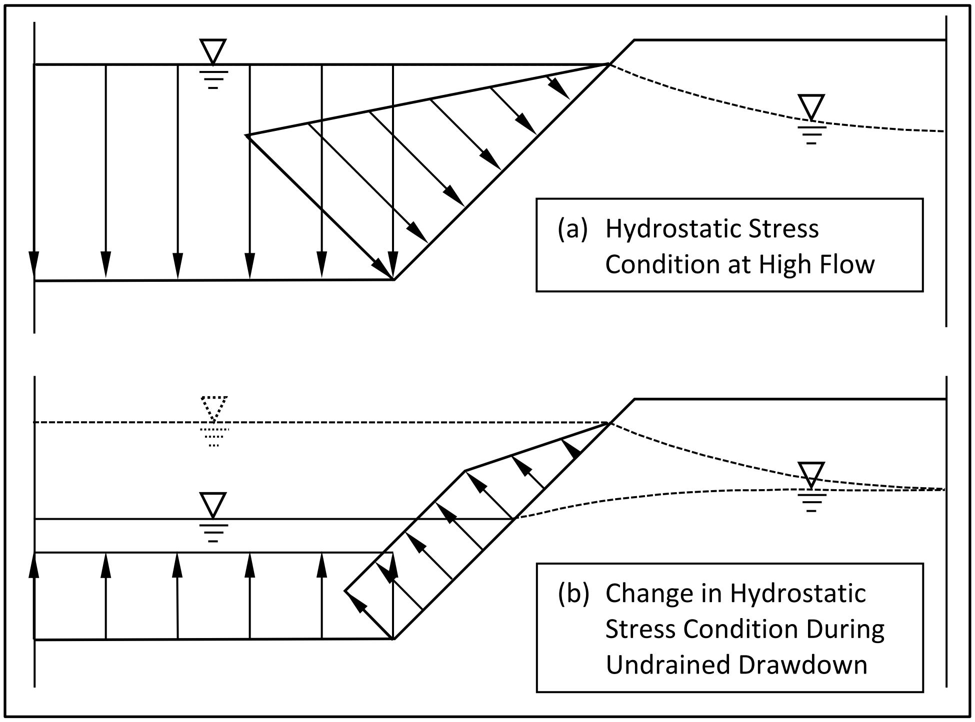


Figure 6.4.5-1 - Drawdown Scenario

6.4.6 Level of Accuracy When Reporting Factor of Safety

Slope stability computer programs can report the FS of failure surfaces to numerous decimal places. Although mathematically possible, reporting slope stability safety factors, and most other geotechnical safety factors, to this level of accuracy is not realistic for several reasons. First, all limit equilibrium methods make various assumptions in order to calculate the FS. Secondly, limit equilibrium methods analyze slope stability using a two-dimensional approach, but actual slope stability is a three-dimensional issue. Finally, no matter how thoroughly the subsurface is explored with borings and laboratory testing, the subsurface conditions (e.g., stratigraphy, soil parameters, groundwater level, etc.) modeled in the slope stability analyses are only a “best fit” approximation of all the data due to the natural variation of soil and rock deposits.

For Department projects, all computer slope stability analysis must show the calculated FS to two decimal places (nearest hundredth); however, the final FS reported for design must be reported to one decimal place (nearest tenth). To demonstrate the rounding requirements for reporting the final FS to one decimal place, use the following example: the slope stability software generated a FS ranging from 1.25 to 1.34; per typical practice, a 1.3 FS would be reported.

6.5 ANALYSIS OF COHESIVE FOUNDATIONS – UNDRAINED (SHORT-TERM) CONDITION

Slope stability analyses must be performed for the final proposed condition using the drained (i.e., effective or long-term) shear strength parameters of the soil. This applies whether the project foundation soils are cohesionless or cohesive. However, when the project foundation soils contain cohesive soil layer(s), undrained (i.e., total or short-term) slope stability must be considered in addition to long-term stability. This short-term condition occurs during construction (i.e., placement of load) and for a period after construction, until any excess pore pressures have completely dissipated.

Specific guidelines detailing when undrained slope stability analyses are required cannot be provided because the need for undrained slope stability analyses is dependent upon site specific conditions. However, some general guidance is provided below to help determine when undrained slope stability analyses are necessary:

1. Cohesive soil is present in the subsurface and is within the depth of influence of the load. If the cohesive soil is beyond the depth of influence of the load, an undrained condition should not develop. Note that the term cohesive includes a broad spectrum of soil types, including, but is not limited to, clays and silty clays (i.e., A6, A7, CL and CH) and silts and clayey silts (i.e., A-4, A-5, ML and MH).
2. The cohesive soil layer is under-consolidated to slightly over-consolidated. These soils are likely to undergo consolidation settlement and develop undrained strength due to development of excess pore pressures. Heavily over-consolidated soil is less prone to consolidation settlement and generation of excess pore pressures, and generally will have a relatively high undrained and drained shear strength.
3. The cohesive soil is saturated (below and sometimes slightly above the groundwater level), or nearly saturated (i.e., high natural moisture contents). If the soil is not saturated or nearly saturated (i.e., void space in soil is not filled or nearly filled with water), the undrained strength is unlikely to develop; however, this may be dependent upon the magnitude of new load placed on the cohesive material. Note that silts and low clay content silts (i.e., A-4, A-5, ML and MH) are less prone to the development of excess pore pressures. Therefore, they are less likely to experience undrained shear strengths. These high silt/low clay materials are generally only subject to undrained shear strength conditions when loads are applied very rapidly.

The slope stability analyses for the undrained condition are performed by modeling the final construction condition, using undrained shear strength parameters (i.e., $\phi = 0$) for the soil(s) that are anticipated to develop undrained strengths during loading. Effective shear strength parameters are used for soil(s) not expected to develop undrained strengths during construction (e.g., cohesionless soils, heavily over-consolidated soil, dry/moist soil, etc.). The static groundwater level should be used for the undrained analyses, although water/pore pressure only affects the effective shear strength of soil and not the undrained strength (i.e., $\phi= 0$).

When undrained shear strength parameters are based on laboratory test data, and there is a high level of confidence in the test data, a minimum required slope stability FS of 1.3 is required for short-term conditions. It is not possible to accurately correlate SPT N-values to undrained shear strength; therefore, it is required that laboratory testing (i.e., UU and/or CU triaxial shear) be performed to estimate the undrained shear strength of soil. If accurate soil shear strength data is not obtained, the results of the analyses will most likely not reflect the actual field conditions.

If the slope stability FS for the construction condition is inadequate, or there is concern with the adequacy of the undrained laboratory shear strength testing, the design must be modified, or an excess pore pressure analysis must be conducted so that a contract special provision can be developed to control the rate of construction at a pace that precludes failure during construction. Design modifications may include the use of:

- Additional slope stability analyses to estimate the allowable rate of loading, (i.e., excess pore pressure analysis according to Section 6.6)
- Prefabricated vertical (wick) drains, sand/stone columns, or other means to drain pore water more rapidly
- Lightweight materials to construct embankments to reduce driving load
- Deep foundations to transfer load below cohesive material

When controls are required to ensure prevention of failure during construction, special provisions that need to be incorporated into the contract documents that must, at a minimum, include:

- Control of load placement based upon the excess pore pressure analysis performed during design
- Monitoring of pore water pressures in cohesive soil layers with piezometers to assess the acceptable rate of loading
- Monitoring of foundation settlement with instrumentation such as settlement plates, vibrating wire settlement sensors, horizontal inclinometers, or other acceptable means, to compliment monitoring of pore water pressures.

Results of pore pressure and settlement monitoring provide the required information for when construction activities (load placement) may have to be paused according to preset thresholds prescribed in the special provision for control of load placement. These threshold(s) are determined from the excess pore pressure slope stability analysis.

6.6 EXCESS PORE WATER PRESSURE

The previous section addressed slope stability analyses for the undrained shear strength condition. This undrained condition occurs when load is carried by the soil pore water. When load is carried by pore water, it creates excess pore water pressure in the soil. It is termed "excess" because it is in addition to the static pore water pressure from ground water that was present in the soil before loading. When undrained slope stability analyses indicate an unacceptable FS, or when a slope failure occurs during construction that is believed to be the result of excess pore water pressure, multiple slope stability analyses that model the excess pore water pressure will be necessary.

Accurately modeling excess pore water pressure in slope stability analyses can be extremely complex. It is performed in a stepped process mimicking the progression of construction (load placement). The analysis is broken up into stages of construction (e.g., 4 to 6 stages – the higher the fill placement, the greater the number of stages required). Fill is added in the slope stability model in stages consistent with how they will be allowed to be placed during actual construction operations.

For example, in current Department approved software, the B-bar method can be used to model excess pore pressure in the soil layer subject to consolidation settlement. B-bar is a factor from 0 to 1 that is applied to the consolidation soil layer. The B-bar value simply indicates the level of excess pore pressure that has developed (e.g., B-bar value of 0.1 is equivalent to 10% excess pore pressure, 0.2 is equivalent to 20% excess pore pressure, etc.). If the static pore pressure (i.e., observed pore pressure before load placement) is 10 psi, then a B-bar value of 0.2 would equate to a total pore pressure of 12 psi, or 2 psi excess pore pressure. The B-bar value is varied in the slope stability model until the resulting FS is equivalent to a predetermined, acceptable, temporary, construction FS.

The B-bar value, correlating to the acceptable, temporary, construction FS for that modeled phase of construction, sets the tolerable excess pore pressure allowed for the construction phase. If the determined threshold excess pore pressure is ever exceeded for the construction phase, the fill placement (i.e., load addition) is halted until excess pore pressures dissipate to some acceptable level (e.g., 50% of the threshold excess pore pressure, etc.). The process is repeated in the slope stability computer model for each phase of construction, to determine the threshold value of excess pore pressure (i.e., the results in the acceptable temporary FS) for each phase.

Some thoughts to consider include:

1. Consolidation settlement (i.e., dissipation of excess pore water pressure) increases the shear strength of the soil. When slope stability analysis using undrained shear strength indicates an unstable or marginally stable condition, consolidation and stability analyses are needed to determine the amount of consolidation, the FS, and the allowable rate of fill placement.
2. It is often assumed that there is no consolidation during construction, but this is generally not an accurate assumption. Consolidation is just time dependent settlement. The rate is a function of the level of excess pore pressure that is in turn a function of the material permeability and the magnitude of the of the load. The rate of consolidation in the field may be fast enough (especially for silty soils) so that a significant amount of excess pore water pressure dissipation, will occur during construction (during load placement). If calculations are performed assuming no consolidation takes place during construction, but partial consolidation actually takes place, the estimated FS will be lower than the actual FS.
3. For numerous reasons, it is extremely difficult to accurately estimate excess pore water pressures and the rate of pore water pressure dissipation during the design

phase, the actual consolidation properties of in-situ foundation soils, and the undrained shear strength of soil. This is why the excess pore pressure slope stability analysis determines a threshold or allowable excess pore pressure that correlates to an acceptable, temporary FS during construction for each phase of construction.

4. There are very few published case histories of failures of embankments during staged construction when sufficient information exists to be able to determine what method(s) are accurate for estimating slope stability when excess pore water pressure exists.

6.6.1 Comparison of Drained (Effective Stress) and Undrained (Total Stress) Approaches

Excess pore water pressure can be modeled in slope stability analyses using two general approaches. One approach is to use drained (i.e., effective stress) shear strength parameters and include the static and excess pore water pressures. Effective stress calculations are performed by subtracting pore water pressure from the weight of the soil. This reduced weight decreases the mobilized shear strength since the shear strength is dependent upon normal force. The other approach to model excess pore water pressure is to use the undrained (i.e., total stress) shear strength parameters. It is not necessary to include the pore water pressure when using the undrained (i.e., ϕ= 0) shear strength parameters because pore water is not subtracted from the weight of soil for the undrained condition.

There are advantages and disadvantages to both approaches. The effective stress method is probably less complicated to perform compared to the total stress method. Additionally, pore water pressure can be measured during construction and compared with the pore water pressure used in analyses performed during design. If needed, analyses can be performed using the field measured pore water pressures, and necessary modifications to construction can be made. One disadvantage of this method of analysis is the laboratory conditions (i.e., consolidated and drained) are not representative of the field conditions (i.e., partially consolidated and undrained) when a total stress failure occurs.

The main advantage of the total stress analysis approach is the laboratory testing better simulates the field condition of the soil during a total stress (undrained) slope stability failure. One disadvantage to this approach is it can be rather complex with respect to the laboratory testing information needed and the analyses itself.

Based on these advantages and disadvantages, it is recommended that when it is anticipated excess pore water may develop, slope stability analyses should first be performed using the undrained (total stress) shear strength parameters. If analyses indicate that the total stress slope stability FS is inadequate or marginal, effective stress (drained) analyses that include excess pore water pressure must be performed. The effective stress analysis is needed to compare to the total stress analysis results, and to help determine the measures required to construct the project without a short-term slope stability failure. During construction, if pore water pressure is measured and the levels/pressures are higher than was modeled in the design phase analysis, effective stress analyses using the measured pore pressures should be performed to determine if modifications to construction are needed to maintain stability.

6.6.2 Methods to Analyze Excess Pore Water Pressure

Excess pore water pressure can be modeled in slope stability computer programs using various methods. The Department approved software has the capability of using water surfaces and the pore water pressure parameter (r_u) to model excess pore water pressure. In addition, the approved software has the options of using the pore pressure coefficient (B-bar), finite element analysis, and pore water pressure grids. Some of these options are briefly discussed below; the individual program documentation should be consulted for more detailed information and how to properly use these options.

6.6.2.1 Water Surfaces

The Department approved software has the option of using “water surfaces” to model excess pore water pressure. The software includes two “water surface” options, a water table and piezometric lines. The software’s water table option is typically intended to represent the static ground water level of the condition being analyzed. The static ground water level is commonly referred to as the water table. Only one water table can be input into the software, and soil layers must be individually assigned to the water table. Typically, any soil layer or portion of a soil layer that is below the water table will be assigned to it.

The other “water surface” option in software is a piezometric line. This option can be used to represent pore water pressure in a soil layer that differs from the water table, such as from a perched water table, artesian ground water, and excess pore water pressure. Both piezometric lines and a water table can be used in the same slope stability model, and numerous piezometric lines can be used in the same slope stability model. However, a soil layer can only be assigned to either the water table OR a single piezometric line. The pore water pressure from a piezometric line is calculated the same as the pressure from a water table; the pore water pressure from a piezometric line is not added to the pressure from the water table.

Water surfaces are useful for modeling excess pore water pressure, but they do have limitations. An advantage of using water surfaces is water level readings obtained in the field, such as from an open standpipe piezometer, can be directly input into the slope stability model. Additionally, water pressures recorded from a pressure transducer, such as a vibrating wire piezometer, can be converted to a water level and input into the slope stability model. However, in some instances the excess pore water pressure may be too large to model with a water surface. Consider that one foot of embankment exerts a pressure of approximately 0.8 psi (i.e., (120pcf x 1ft)/144)). Assuming the full embankment pressure is transferred to the soil pore water (i.e., no drainage and no stress distribution/reduction), the pore water pressure increase for each foot of embankment placed would also be approximately 0.8 psi. Since the unit weight of water is approximately half that of typical embankment, approximately two feet of water “head” is needed to model the possible excess pore water pressure produced from one foot of embankment (i.e., (62.4pcf x 2ft)/144).

Therefore, for projects where the water level to model pore water pressure exceeds the ground surface level, the use of water surfaces is not appropriate. For these and other instances, the slope stability software provides the option of using the B-bar method.

6.6.2.2 B-bar Method

The B-bar method is a very useful tool for modeling excess pore water pressure. This method can be used in conjunction with other water pressure features within the software. For example, the “water table” option can be used to define the static water pressure from groundwater, and the B-bar method can be used to model additional pore water pressure from loads such as embankments or footings. To use this method, a B-bar coefficient ranging from 0 to 1 must be input for each soil layer that is expected to develop excess pore water pressure. Additionally, the load, $\Delta\sigma$, (i.e., embankment, external load, or seismic load) that will create the excess pore water pressure (Δu) must be identified. The excess pore water pressure is calculated by multiplying the load identified as causing excess pore water pressure by the B-bar coefficient (i.e., Δu = B-bar x $\Delta\sigma$).

If a B-bar coefficient of 1 is used, the excess pore pressure applied to the layer(s) identified will equal the pressure from the load that was identified as creating the excess pore water pressure, such as in the case when no drainage or stress reduction/distribution occurs. If pore water pressure is measured in the field, the B-bar coefficient can also be used to approximate the field measured pressure. Note that if the soil layer is relatively thick and the pore pressure varies throughout the layer, it may be beneficial to use two or more thinner layers so that the pore water pressure can be more accurately modeled.

Note that the B-bar method appears to be a simplification of the method proposed by Skempton in 1954. Skempton’s method includes pore pressure parameter B, but this method also includes pore pressure parameter A. The pore pressure parameter A can range from -0.5 to 1.5. Although difficult to accurately estimate, these pore pressure parameters can be estimated from laboratory tests.

6.6.2.3 Pore Pressure Parameter (r_u)

The pore water pressure parameter (r_u) is another option for defining excess pore water pressure in soil layers, but it is generally not as useful as the previous options discussed. The pore water pressure parameter is defined as the ratio of total pore water pressure to total overburden pressure, or alternatively defined as the ratio between the total upward force from water pressure to the total downward force from overburden pressure. Mathematically it can be written as:

$$r_u = \frac{u}{(\gamma_s \; z)}$$

where:

u = pore water pressure at depth z (psf)

γ_s = total unit weight of soil (pcf)

z = depth of point below ground surface (ft)

Except for the case of fully saturated, homogenous soil, r_u will vary with depth. For fully saturated, homogenous soil, r_u will be approximately 0.5 since the unit weight of water is approximately one half the unit weight of soil. Soils that are not fully saturated will have a r_u value of less than 0.5. The use of the pore pressure ratio is beneficial for relatively simple slope stability problems with saturated, homogenous soils. There is a possible problem with using r_u to define excess pore water pressure in a soil layer. Excess pore water pressure imposed by an embankment load is believed to be proportional only to the increase in overburden that the embankment represents, but r_u would give pore water pressures that were proportional to the total overburden for the soil layer considered to the embankment surface. Therefore, this method is not preferred for use by the Department when modelling construction sequencing for embankments subject to excess pore water pressure build up.

6.6.3 Requirements for Analysis with Water

When groundwater is involved, any appropriate methods may be used in support of the final design analysis. However, to ensure slope stability analyses involving groundwater are interpreted properly, the following requirements for methods are to be followed unless specified otherwise. Any proposed final design method that differs from the original design method must be approved by the CGE. To obtain approval, thorough documentation must be submitted for any proposed deviations from these methods, including supporting technical justification that the proposed alternate approach is both required and appropriate for the specific site conditions and circumstances. The required standard methods of analysis are indicated in Table 6.6.3-1.

Table 6.6.3-1 – Required Slope Stability Analysis Methods

Condition	Analysis Method
Long-Term Drained Conditions	Effective Stress Analysis
Short-Term Undrained Conditions w/o Monitoring for Excess Pore Pressure	Total/Undrained Stress Analysis (ϕ= 0)
Short-Term Undrained Conditions with Monitoring for Excess Pore Pressure	Staged B-bar Analysis

6.6.4 Piezometers

When excess pore pressures are a concern, piezometers need to be installed before construction of the embankment, foundation, etc. Plans and specifications for piezometer installation and automated instrumentation (e.g., vibrating wire transducers, data loggers, etc.), if used, are typically included in the construction contract documents. The piezometers need to be installed in the layer(s) where excess pore water pressure may develop. Generally, the piezometers should be placed in the middle of the layer, but if the layer is relatively thick piezometers may need to be placed at various levels. Static ground water levels should be recorded in the piezometers before construction to establish baseline water levels. Pore water pressure measured by the piezometers during construction are used to control the rate of fill placement based on the results of excess pore pressure stability analyses performed during design. Furthermore, the piezometers will aid in the evaluation of the status of consolidation settlement.

6.6.5 Modeling Excess Pore Water Pressure

For an example of how excess pore water pressure can be modeled, assume an embankment 50 feet high is proposed to be constructed over soft, saturated clay that is 20 feet thick. The clay layer is located 20 feet below ground surface, and silty gravel overlies the clay. The static ground water level is approximately 10 ten feet below ground surface. Consolidation settlement testing indicates that the clay will develop excess pore water pressure during loading. Undrained (total) and drained (effective) shear strength testing was conducted to estimate the shear strength properties of the clay. Parameters for the example are as shown in Table 6.6.5-1. A cross-section of the example is presented in Figure 6.6.5-1. Note that for this example, the Rocscience software, SLIDE2, was used as a tool to demonstrate the process; however, this process is not specific to SLIDE2 software.

Table 6.6.5-1 – Example Effective Stress Parameters

Material	Wet Unit Weight, (pcf)	Saturated Unit Weight, (pcf)	Cohesion, c, (psf)	Friction Angle, ϕ, (degrees)
Embankment Fill	125	N/A	50	32
Silty GRAVEL	125	130	0	34
CLAY	115	120	400	10
Rock	145	N/A	1000	45

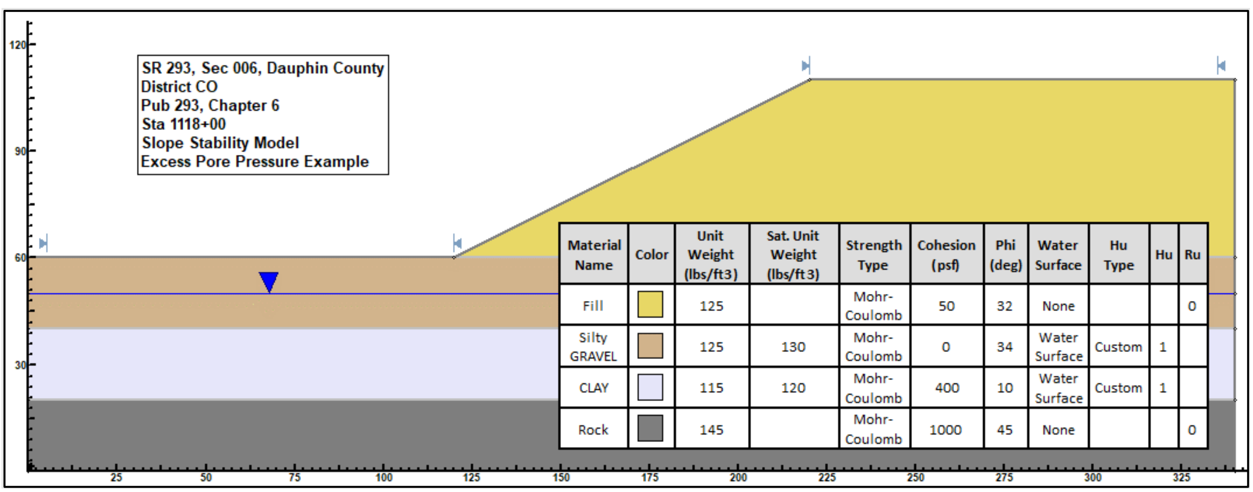


Figure 6.6.5-1 – Slope Model for Excess Pore Pressure Analysis

1. First check the long term drained (effective stress) stability of the proposed embankment. Slope stability analysis of this slope results in a minimum factor of safety (FS_{min}) = 1.48 meeting the 1.5 minimum factor of safety requirement for long term conditions. See Figure 6.6.5-2.

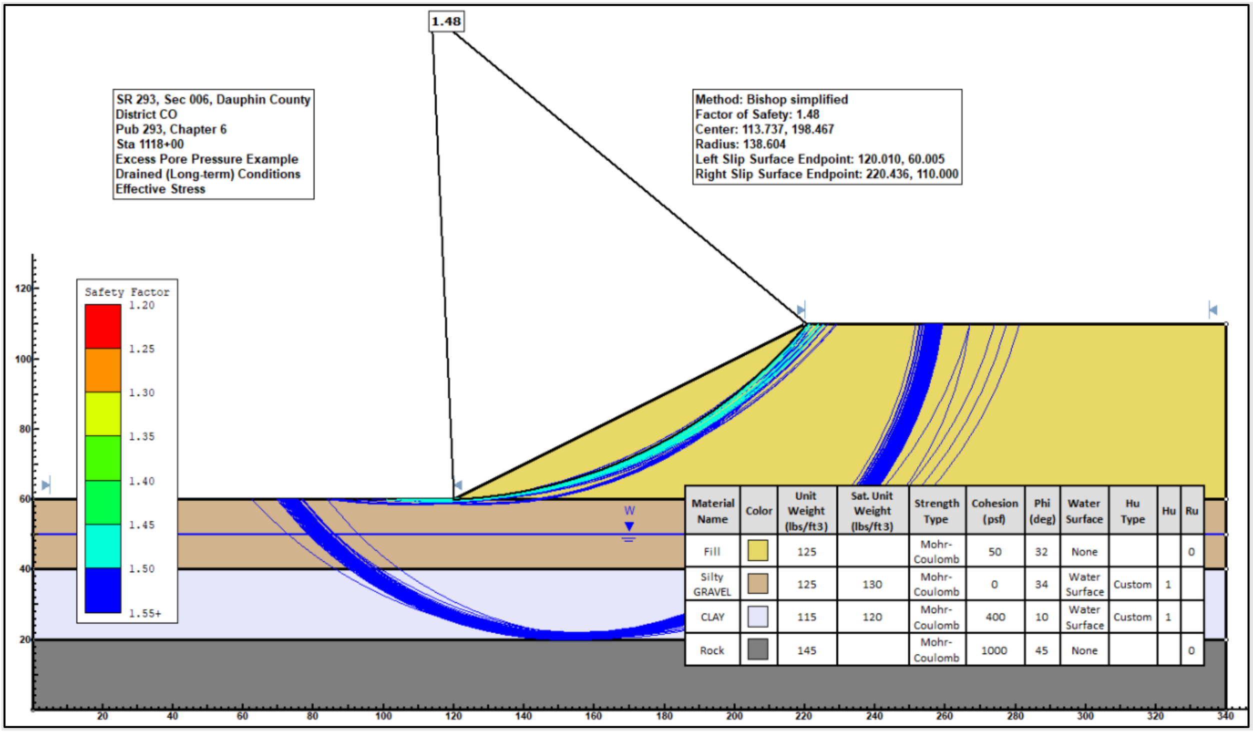


Figure 6.6.5-2 – Results for Drained (Effective Stress) Analysis

2. Perform slope stability analysis using the total (undrained) shear strength parameters of the clay to simulate the short-term construction conditions (i.e., excess pore pressures have developed and have not had sufficient time to dissipate or drain by the end of construction). As discussed earlier, pore water pressure is ignored when total stress (i.e., $\phi = 0$) parameters are used; therefore, both static and excess pore water pressure are ignored for the clay layer. The static water level is required to be defined for layers that are modeled using effective strength parameters (i.e., the silty gravel layer and the clay layer) so that the saturated unit weight is used in the analysis. In the slope stability computer model this is accomplished by entering all layer boundaries and parameters, and then indicating no water table in the clay layer. Keep the saturated unit weight for the short-term undrained loading condition.
3. Assume that the entire embankment is constructed relatively quickly and with no or little drainage (i.e., consolidation) occurring during construction. In other words, model the 50-foot high embankment and use the total (undrained, $\phi = 0$) shear strength parameters of the clay to simulate the end of construction condition. Because the other soil layers are considered to be relatively free draining, development of excess pore water pressure is not expected; therefore, effective shear strength parameters apply to these layers. The FS for this condition is FS = 0.99 (see Figure 6.6.5-3). This is below the minimum required FS of 1.3 for temporary construction conditions.

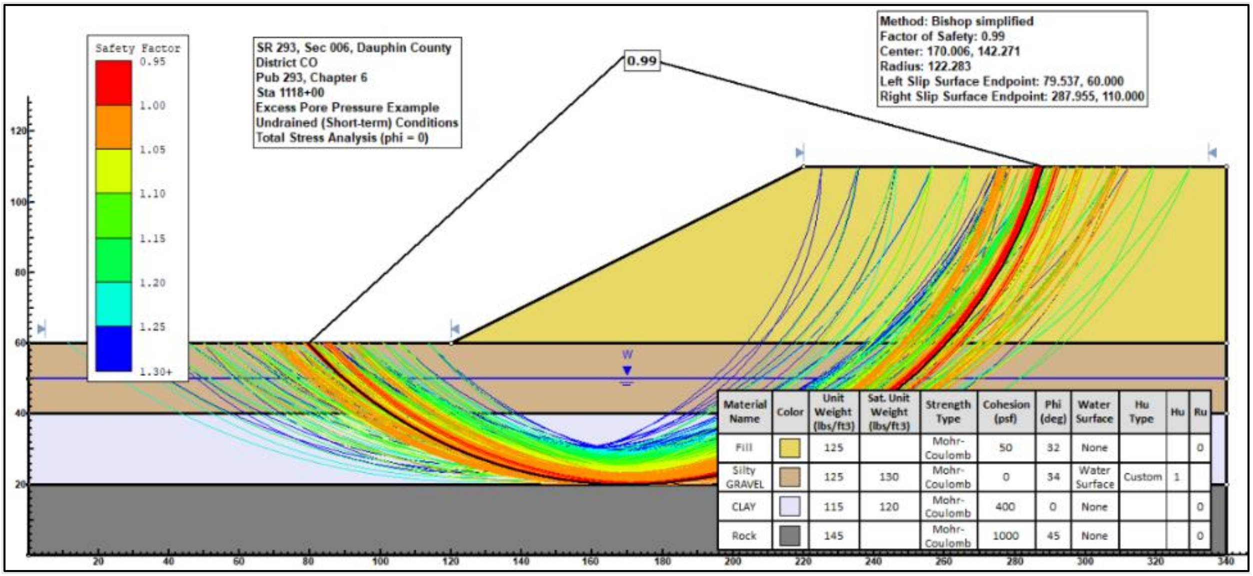


Figure 6.6.5-3 – Results for Undrained (Total Stress) Analysis

4. If the minimum required FS had been achieved for the undrained analysis above, that would indicate the embankment could be constructed without an undrained slope stability failure. Note that if the short-term condition did provide an acceptable FS, depending upon the confidence level in the undrained analysis model, it is prudent to perform the additional analysis outlined in the steps below, monitoring pore water pressure during construction to better ensure the short-term stability of the embankment is maintained.
5. When an unacceptable or marginal FS results from the undrained (short term/total stress) analysis (i.e., Step 2), additional analyses must be performed using the undrained shear strength of the clay. These analyses should model intermediate embankment heights to determine the approximate embankment height at which the minimum required slope stability FS is achieved. This is where B-bar analysis becomes particularly useful. Stability analysis using the B-bar method for the example above is included in the Department's SLIDE2 User's Manual. A short description of the analysis is provided here; however, see the SLIDE2 User's Manual for additional details. Essentially the embankment is analyzed in stages using the computer model. For the 50-foot-high example embankment, the analysis was broken down into 5 stages of 10 foot each. The computer analysis indicates that the example embankment (with the indicated foundation soil and water conditions) can be constructed to a height of 20 feet maintaining the minimum required 1.3 FS. From a height of 20 to 30 feet, embankment construction can continue until the excess pore pressure is 72% above static or preconstruction levels (1.72 x static pressure) for any of the installed piezometers used for monitoring pore pressure. At this point, construction of the embankment would have to be halted until the excess pore pressure dissipates to some predetermined acceptable level. From 30 to 40 feet, embankment construction can continue until the excess pore pressure is 50% above static levels (1.50 x static

pressure). And from 40 to 50 feet, construction can continue until the excess pore pressure is 35% above static levels (1.35 x static pressure).

For the example provided, piezometers would be required to monitor pore water pressure in the clay layer during construction. Additionally, language would have to be provided in the specification that limits the amount of pore water pressure that can develop in the clay layer during construction of the fill. Based on the results of analyses for the slope in question, the specification may be able to allow construction to some permissible height without any pore water pressure restrictions; however, once the allowable height is reached, the specification would require the excess pore water pressure to be at or below permissible limits (that provides an acceptable short-term FS) to complete construction to full embankment height.

When preparing the specification, ensure that it satisfies the slope stability analysis, is not overly restrictive/conservative such that it will unnecessarily increase construction time/cost, and that it is as straight-forward as possible. If the construction schedule does not allow the necessary time for pore water pressure to naturally dissipate, other measures (e.g., vertical drains, etc.) may be required to limit the development of excess pore pressures.

6.7 SURFICIAL FAILURE SURFACES

Surficial failure surfaces are slope stability failure planes that are at a very shallow depth below the surface of the slope and nearly parallel to the slope face. These failure planes are very common when analyzing slopes constructed with cohesionless (i.e., cohesion equal zero) soils. The shear strength of cohesionless soil is directly proportional to the normal force applied to the soil; therefore, at shallow depths (i.e., low normal force) even "competent" cohesionless soil can have relatively low shear strength. Although the shear strength may be relatively low, the driving/destabilizing force will also be relatively low at these shallow depths. As a result, low surficial factors of safety may exist (less than the required minimum).

For example, consider the infinite slope analysis method for estimating the slope stability FS, which is applicable to infinite slopes (about 15 feet or higher) constructed with cohesionless soil. For a dry slope, the FS is estimated by dividing the tangent of the internal friction angle of the soil by the tangent of the slope angle, as measured from the horizontal. Therefore, for a 2H:1V slope (26.6 degrees) constructed with soil having an internal friction angle of 30 degrees, the estimated slope stability FS using the infinite slope method is approximately 1.2. This FS is below the required values indicated above for the permanent/final condition, but it is not necessarily unacceptable.

Surficial failure surfaces often have the lowest safety factors based on slope stability analyses. Although these surfaces must be identified and considered, they are typically not the most critical failure surfaces for several reasons, including:

1. Soil slopes are typically vegetated. Vegetation will provide some surficial cohesion in the soil that improves stabilization, which is typically ignored during the analysis.

2. Soils often have some minimal cohesion, which is also typically ignored during analyses. Even minimal cohesion improves surficial slope stability.
3. Shear strength of soil is low near the surface since there is very little overburden pressure to mobilize the frictional component of soil shear strength. Although the surficial shear strength of cohesionless soil may be relatively low, the driving force associated with the surficial failure surface is also low. Consequently, the risk of failure is low, and marginal factors of safety can be tolerated for surficial failure surfaces.
4. The consequence of a surficial failure will typically be low, amounting to a maintenance issue of regrading and vegetating the slope. A costly slope treatment (e.g., rock veneer, slope flattening, etc.) to achieve the minimum required FS may not be warranted.

There are a few ways that the analyses can be performed to avoid these surficial surfaces from appearing. One approach is to model the surface soil on the slope with some minimal cohesion (i.e., less than 25 psi), if appropriate for the soil type. Another approach is to force the failure surface a certain distance, usually three feet or so, below the face of the slope. Most computer programs have this feature. A third approach to help avoid surficial failures from appearing is to force the entry/exit of the failure surface a few feet behind the crest of the slope and/or a few feet beyond the toe of the slope.

For Department projects, surficial slope stability failure surfaces must be investigated. If surficial failure surfaces do not meet the minimum required safety factor(s) presented in Section 6.4, they must be discussed in the appropriate geotechnical engineering report. If reasonable justification can be provided, surficial slope stability safety factors lower than the minimum required may be appropriate.

Typically, as long as surficial factors of safety are greater than or equal to 1.2 (Surficial $FS_{min} \geq 1.2$), and the required minimum FS is obtained rapidly with depth, such conditions are determined to be acceptable so long as both short- and long-term erosion protection measures are employed (i.e., temporary erosion control and long-term vegetation or slope armoring).

6.8 SURCHARGE LOADS

Surcharge loads on top of or within slopes create a driving (i.e., destabilizing) force that reduces the slope stability FS and, therefore, must be included in slope stability analyses. Surcharge loads can be either temporary or permanent. The surcharge load from vehicle/truck traffic is the most common with respect to roadway embankments. Traffic surcharge is considered a permanent load and must be modeled using a uniform strip load of 360 pounds per square foot (psf). The traffic surcharge must be applied over the full width of the traveled roadway and the paved shoulders. Structures supported on spread/shallow foundations including highway structures (e.g., bridges, retaining walls, noise walls, etc.) and buildings that are located on top of or within slopes also create a permanent surcharge load. Loads from shallow foundations (i.e., the actual/service limit state bearing pressure) must also be included in slope stability analyses. Waste materials (i.e., soil and rock) may be another source of permanent surcharge load that may have to be considered if placed near a slope.

Temporary surcharge loads located on top of or within slopes that should be considered can include, but is not limited to, stockpiled construction materials (e.g., aggregate, embankment, etc.), temporary footings for bridge erection, and heavy machinery such as cranes. While these loads may be temporary, the slope stability analyses may or may not have treated the load(s) as temporary. Refer to Section 6.4.4 for guidance of required FS and whether a temporary loading condition is treated as temporary or permanent relative to required FS. Note that if a consolidating layer is present within the area of influence, the effects of surcharge loads contributing to the generation of excess pore water pressure must be a consideration in the slope stability analyses. If the slope stability analyses indicate an unsatisfactory FS, provisions may have to be included in the contract documents that prohibit or limit the contractor from applying loads on the slope.

6.9 SLOPES IN OR ADJACENT TO WATERWAYS

For slopes in or adjacent to waterways, it is important to include protection for the toe of the slope from erosion. In the absence of any other requirements, top elevation of the slope armament must exceed the appropriate design flood elevation by a minimum of five feet (where practical) or extend to the top of the slope in situations adjacent to a flood plain. The slope armament must be designed and sized to withstand flow velocities anticipated for the design storm and flooding. **The armament must not be considered as an enhancement to slope stability during analysis, but as a stand-alone feature solely for slope protection.** Any similar treatments required for slope stability because of the water environment or for high water conditions, must be considered and designed separate and in addition to required slope armament.

6.10 BACK ANALYSIS

When a slope failure occurs, an analysis is usually performed in an effort to identify probable causes. By using a known or assumed failure surface, back analysis can be used to help verify the validity or accuracy of slope stability models. Back analysis is extremely useful when analyzing slope failures, and it can also be used for the analysis of existing slopes. However, back analyses must be used with caution and only under the correct circumstances.

Back analysis is often performed on a slope for a given set of conditions so as to estimate the shear strength that exists or is required for those conditions. Back analysis has a number of useful applications. Some of the applications of back analysis include:

1. Estimating the shear strength of soil for an existing slope failure when the subsurface profile and location of the failure surface are well defined
2. Confirming shear strength test results conducted on soil samples from an existing slope failure when the subsurface profile and location of the failure surface are well defined
3. Estimating the location of a failure surface when the subsurface profile is well defined, and a reasonable estimate of soil shear strength is available

4. Assessing if proposed construction activity will impact an existing slope with reasonably defined subsurface profile, but unknown state of shear strength (i.e., an existing slope that appears stable but there is no calculated FS and no shear strength testing available)

When conducting slope stability to estimate the FS against failure for a known or proposed set of conditions, conservative assumptions (e.g., low shear strength, high water table, etc.) will result in a conservative analysis. The opposite is true for back analysis. Unconservative assumptions provide more realistic results. In back analysis, conservative assumptions of material shear strength (low estimated shear strength), low values for material unit weight, high groundwater levels, or the inclusion of transient loads such as a live load surcharge, will result in very unconservative, and sometimes unrealistic, results (i.e., higher than actual or possible FS).

To better illustrate how back analysis is used, consider the impact of a live load surcharge when conducting back analysis on a failed slope just before failure. Including the live load surcharge increases the required shear strength to get to a condition of equilibrium (i.e., FS=1.0), or the condition immediately before slope failure. But a live load surcharge is a transient load and is not always there. Therefore, if the live load surcharge is included in the back analysis, the shear strength of the material will be overestimated for the failure condition. In back analysis, only permanent loads or conditions should be included in the slope model when attempting to assess conditions just before failure. The same impact would result if the water table elevation is estimated too high. Higher shear strength values will be required to achieve equilibrium (i.e., FS = 1.0). The resulting slope model just before failure would be unconservative because the shear strength parameters were over estimated.

Back analysis is intended to be used in conjunction with good quality subsurface information meeting the requirements in Section 6.2. Back analysis must not be used in place of obtaining subsurface information from borings and laboratory soil classifications. If absolutely necessary, back analysis can be used to estimate soil shear strength for well-defined subsurface conditions (i.e., borings and lab classifications); however, this must be reflected in the required minimum FS. Note that it is always preferred to have laboratory shear strength testing and inclinometer data indicating the location of the failure plane (for failed slopes), so that back analysis is primarily used as a model verification tool as opposed to a primary design tool (i.e., the model is used estimate shear strength parameters for design). Once all available subsurface information is obtained and the slope stability model is developed, back analysis can be used to help determine if the model appears reasonable and help better define parameters that may be difficult to estimate otherwise.

Back analysis may be more difficult to use for a stable slope when compared to a failed slope (i.e., landslide). A failed slope will typically have a FS of 0.9 to 1.0 since the slope was most likely stable at one point until a change in conditions caused the slope failure. Additionally, the failure plane generated by the slope stability analysis should be very similar to the actual failure plane identified by field observations (e.g., head scarp, tension cracks, toe bulge, etc.) and instrumentation (i.e., inclinometers and piezometers).

Conversely, an existing stable slope can have a FS of a much broader range (i.e., > 1.0), and there is no field failure plane to compare to the computer-generated failure plane. Therefore, when applying back analysis to an existing stable slope, or when attempting to estimate shear strength parameters, an assumed existing FS of 1.0 must be used. Such situations that generally result in a conservative analysis. However, there are situations when a conservative analysis can still provide an acceptable result, saving valuable design and/or construction resources.

For example, assume a landslide occurred and thorough subsurface laboratory and instrumentation programs were completed. The computer slope stability model was generated, and the results indicated a critical failure surface similar to the surface observed in the field; however, the FS of the critical surface was 1.2, indicating failure should not have occurred. The analysis used piezometer data to model groundwater conditions, but it was believed that levels may be higher at times due to seasonal fluctuations and precipitation.

Since the other parameters (e.g., shear strength, unit weight, stratification, etc.) were based on reliable field and laboratory test results, a back analysis could be performed by increasing the groundwater level until a FS of 1.0 was achieved. If the groundwater level associated with the FS of 1.0 appeared reasonable, then it could be assumed that the elevated groundwater level triggered the landslide. This elevated groundwater level should then be used for the landslide remediation design.

Back analysis is not recommended when values for more than one parameter are not well known. For example, assume a back analysis is performed for a landslide. However, instead of having good quality laboratory and ground water data, shear strength values based on laboratory classifications, and a ground water level based on 0-hour ground water readings were used to develop the slope stability model. Results using this model yielded a failure plane with a FS of approximately 1.0 that was similar to the failure plane observed in the field, and the back analysis indicated that the slope stability model was reasonable.

This slope stability model was then used to design the landslide remediation, and the remediation would concentrate on improving either loads on or resistance of the system, because ground water was believed to be relatively low. Contrary to the analysis, the actual site conditions included a higher shear strength value and a much higher ground water level compared to what was used in the back analysis. Consequently, the remediation could be unsuccessful since it did not include provisions to lower the groundwater level at the site, and the focused increase in resistance or reduction in load, may prove insufficient.

Finally, consider a situation where back analysis is used to assign soil strength parameters for an apparently stable slope where proposed construction is planned. Available subsurface information is limited to historical data from the original design and construction of existing facilities consisting of only original descriptions of subsurface materials. No groundwater elevation is provided, but an approximate groundwater level can be inferred from the material descriptions. A conservative back analysis using an assumed low groundwater elevation, and ignoring any temporary or transient loads, is conducted to determine soil strength parameters. A FS less than 1.0 was the target for the back analysis. The intent of the proposed construction is to match existing conditions, leaving the site with the same or increased level of stability.

Although it is believed that since the location has no signs of instability, and the soil strength parameters determined from back analysis appear conservative for the available material descriptions, an absolute or actual FS for the existing stability condition cannot be quantified due to the lack of any available laboratory shear strength testing or actual groundwater elevation. Although the estimated groundwater elevation, back analysis, and resulting assigned shear strength parameters are qualitatively evaluated as conservative, there is no actual data to conduct a supporting quantitative analysis.

For such a situation, it is important that the proposed construction be designed with the analysis conducted with some quantifiable level of conservatism. In other words, the proposed construction cannot be designed to a FS just above 1.0 to exactly match existing conditions. The target FS for the proposed construction must be set at some appropriate level consistent with both the risks and consequences of failure. Such a target FS value must be assigned based upon specific site conditions, including the complexity, value of the proposed construction, and the risk and consequences of failure. It is recommended that in no case should the new target values be less than a FS = 1.1, or a 10% increase in the ratio of driving to resisting forces.

6.11 INVESTIGATION OF SLOPE FAILURES

The detailed process for investigating, analyzing and remediating slope failures is beyond the scope of this publication, but several publications are available to assist with direction on how to perform this work. For purposes of providing a generalized, typical process, the following should be considered to analyze and remediate slope failures.

1. A detailed **field reconnaissance** is required when investigating slope failures. Visual observations of landslide features provide extremely valuable information that can help identify the cause(s) of slope failures and that can be used in slope stability analyses (e.g., location of head scarp, toe bulge, seeps, hummocky ground, bent/tilted trees, deflected guide rail, etc.). It is important to accurately locate any landslide features on project mapping. If they cannot be located accurately during the field reconnaissance, the features should be marked with flags or stakes and surveyed at a later time. Note that as there are advantages to performing field reconnaissance before the review of available information, these steps can be switched, or additional site reconnaissance can be performed, if needed, based on the findings from the review of available data. Refer to Chapter 2 for more information pertaining to field reconnaissance of a landslide.
2. It is important to **review available information** for the project area. This information may help identify the cause(s) of the landslide, and/or may provide information that could be useful for analysis of the landslide. Available information may include site specific information (e.g., boring information, previously prepared geotechnical reports, roadway plans/section, etc.), published geologic reports, topographic maps, mine maps, aerial photography, and more.
3. A detailed **subsurface exploration** is needed to properly analyze slope failures. Significantly more effort is required when compared to a typical roadway exploration program. Cross-section borings, including borings above and below the head scarp, within the sliding mass, and below the landslide are typically

needed. Additionally, borings ahead and back station of the sliding mass are typically recommended to ensure the limits of the unstable slope. The purpose of the borings is to characterize the subsurface conditions, collect soil samples for laboratory testing, and install instrumentation. Other types of sampling/testing in addition to split-spoon sampling, standard penetration testing, and Shelby tubes may also be beneficial. These could include cone penetrometer testing, in-situ testing (e.g., vane shear, etc.), test pits, and geophysics.

4. **Laboratory testing** is critical for the study of slope failures. Similar to the subsurface exploration program, the laboratory testing program for landslide investigations is typically more comprehensive compared to a typical roadway testing program. It is important to ensure enough testing is required so that an accurate subsurface model can be developed for use in the stability analysis. Laboratory testing will typically include natural moisture content, grain-size distribution, liquid and plastic limits, unit weight, and shear strength testing. Depending upon the soil types, shear strength testing may include unconfined compression, peak direct shear, residual direct shear, unconsolidated-undrained triaxial shear, and consolidated-undrained triaxial shear with pore water pressure measurement. Where possible and when appropriate, undisturbed samples should be used for shear strength testing. A general rule is that when the material will be excavated and/or moved and recompacted, then remolded samples are appropriate for lab testing. Otherwise, undisturbed samples are preferred when possible.
5. **Instrumentation** provides valuable and necessary information for the analysis and remediation of slope failures. Instrumentation consisting of inclinometers and piezometers are typically required for landslide investigations. Inclinometers provide the location/depth of the sliding plane. Inclinometers should generally be installed in cross-section within and near the top and bottom of the landslide. Depending on the length of the failure plane, an additional inclinometer near the center of the slide mass may be warranted. Since groundwater typically plays role in slope failures, piezometers should be installed to help estimate the groundwater level. Vibrating wire piezometers, either grouted in place or placed inside of an open standpipe type piezometer, are typically the most beneficial. These can be connected to a data logger that periodically (e.g., hourly, daily, etc.) records the groundwater level to identify groundwater fluctuations that may result from precipitation or other events.
6. The sum of information and data obtained and collected in Items 1 through 5 above is used to develop a preliminary slope model. The various components of information and data are used to build a virtual representation of the slope, usually both before and after the slope failure. Parameters used in the model must reflect and be representative of the information and data obtained. After building the virtual model, it is tested by conducting computer slope stability analyses. A “back analysis” can be performed to see if the modelled conditions were able to produce a pre-failure slope stability condition close to equilibrium (i.e., FS ≈ 1.0), resulting in a critical failure surface representative or close to the surface observed and/or indicated by instrumentation. If a FS just below 1.0 is obtained, and the critical surface obtained in the analysis is in close proximity of the field observed and/or instrumentation indicated failure surface, then the model can be considered

to be reasonably representative of pre-failure conditions; thus, can be used to help develop a remedial design. If stability "back analysis" does not result in this level of correlation, then the proposed model needs to be reevaluated and/or site investigations may be required. If investigating an existing slope that has not failed, but is being subjected to changing loading conditions, a similar process can be used to help assess potential impacts of the new loading conditions.

7. Once the slope stability model is believed to be representative of the actual field conditions, the slope remediation design can begin. Two broad options when it comes to addressing unstable slopes are reducing load or increasing resistance. Examples of reducing load include removal of material driving the slide, use of lightweight fill, and drainage improvements. Examples of treatment that increase resistance include earth berms or buttresses, slope flattening, improving drainage, retaining structures, deep foundation elements (e.g., piles, drilled shafts, anchors, etc.), and deep ground improvement. Note that drainage is mentioned under both load reduction and increasing resistance. That is because water can add to driving forces by saturating and increasing the weight of a soil mass, but also decreases soil shear strength by reducing the effective stress.

6.12 MODELING EXISTING FAILURE SURFACES

Occasionally, an existing failure plane or surface is present in the subsurface of an existing slope. These are failure surfaces on which movement has occurred in the past (i.e., inactive or dormant failures), or where movement is ongoing (i.e., active failures). These surfaces could be naturally occurring deposits, such as varves or other weak strata typically consisting of clay, or they could be the result of added loads (e.g., an embankment, structure, etc.), an excavation, or remnant from an ancient slope failure. The presence of slickened sides observed in soil samples, visual observations of slope movement, or data from instrumentation could be an indication that a failure surface is present on a project site.

Failure surfaces tend to develop in clayey soils because these generally have a lower shear strength compared to granular soils. Additionally, as movement occurs within clayey soils, the platey clay particles align themselves along the direction of movement that causes a decrease in the shear strength along the failure plane. When movement occurs within granular soils, the soil grains reorient themselves with little to no reduction in shear strength. Clayey soils also present more visual evident of active or dormant slide planes

When an active or inactive failure surface is present in the subsurface of a project site, it is most likely the weakest layer that will control the slope stability of the project. Therefore, it is imperative that the surface be identified/located, sampled, and tested. The surface can be located by visual observations, borings, and/or instrumentation, with samples obtained by borings. Although the actual failure surface will likely not be visible within the soil samples, it is necessary to obtain samples of the stratum in which the failure surface is located. When possible, undisturbed soil samples should be obtained. Lastly, laboratory testing, including residual direct shear strength, must be performed on the stratum in order to have the necessary information to develop the slope stability model. Residual direct shear testing is discussed in detail in Chapter 4 of this publication.

The existing failure surface must be included in the slope stability model. For planar failure surfaces, this surface can be represented by a “band” that encompasses the failure surface. Although the failure surface is generally envisioned as being a discrete surface passing through the soil, the layer or layers containing the surface will have to be modeled accordingly with the appropriate shear strength parameters. The failure surface will generally have to be modelled as a discreet soil layer. Note that the failure surface often passes through more than one soil layer. Each distinct cohesive soil layer (if such distinctions can be made) that the failure surface passes through should be modeled with residual shear strength parameters. It is often the case that significant distinctions between these layers cannot be made. In such cases, a composite residual shear strength material model should be used that closely matches field observations of the failure surface location in a back analysis.

6.13 SEISMIC SLOPE STABILITY ANALYSIS

Seismic events (i.e., earthquakes) are relatively infrequent in and around Pennsylvania, and ground accelerations in Pennsylvania from seismic events have generally been low. Consequently, in most cases seismic slope stability will not control the design of embankments or structures. Therefore, seismic slope stability analysis is not required for Department projects. However, for projects located in the eastern part of the state where ground accelerations are highest, and where high embankments, high retaining walls, large, multipart, and/or critical structures are proposed, seismic slope stability may be required by the Department. If seismic slope stability analysis is deemed necessary for a project, it must be clearly stated in the project specific scope of work, or the original scope of work must be amended with the specific seismic analysis requirements.

The discussion below is based on information from the publication FHWA-NHI-11-032, GEC No. 3, titled “LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations”, dated August 2011 (Revision No. 1). This publication should be consulted for a more detailed discussion of seismic slope stability analysis.

Ground shaking from a seismic event (i.e., dynamic forces) will increase the soil slope destabilizing force and result in reduction of soil shear strength, thus lower the slope stability FS compared to the static conditions. However, due to the extreme, infrequent nature of seismic events, a FS lower than required for the static condition is acceptable for the seismic condition. Consequently, due to the relatively low ground acceleration coefficients associated with Pennsylvania, a properly designed and constructed slope that meets static slope stability FS requirements will, in almost all cases, meet the seismic slope stability FS requirement.

Seismic slope stability analysis is typically performed using a pseudo-static limit equilibrium approach. Similar to the static analysis, when seismic slope stability analysis is required, it must be performed using the Department approved slope stability software. Also, similar to the static analysis, either the Simplified Bishop or Simplified Janbu method must be used for the seismic analysis.

The limit equilibrium pseudo-static stability analysis method is termed such because this method uses conventional limit equilibrium analysis and includes a static horizontal force to represent the inertial (dynamic) force of the seismic event. A seismic coefficient is the static force that is used to represent the inertial force. The seismic coefficient is typically some percentage of the peak ground acceleration from an earthquake of a given return period.

An important consideration in employing the limit equilibrium approach to seismic slope stability analysis is that the rate of loading during the earthquake is relatively fast. For this reason, in most cases undrained total stress strength parameters should be used in the seismic stability model if the soil is saturated, rather than drained, or effective stress parameters. The static undrained strength is generally considered to be an upper bound on the undrained strength that should be used in a seismic stability analysis.

Design Manual 4 (2015), in conjunction with AASHTO LRFD Bridge Design Specifications (2014), addresses seismic design for structures. AASHTO recommends that structures be designed for a more rigorous displacement-based method. Also similar to the static analysis, a displacement-based method may be used to supplement the Department approved software analysis if approved by the CGE.

Based on these publications, seismic slope stability analysis is not required for single span structures. Additionally, since Pennsylvania is located in Seismic Zone 1 (i.e., acceleration coefficient < 0.15), seismic analysis is not required on multiple span bridges unless they are located in Site Class E or F.

Note that soil liquefaction caused by a seismic event is a separate issue from seismic slope stability and must be addressed independently. The above mentioned FHWA publication includes discussion on subsurface conditions that are prone to liquefaction, and guidance on performing liquefaction analysis. Site specific effects of soils affecting bridge behavior are addressed in AASHTO LRFD 3.10.3. Soils meeting the criteria for Site Class E or F may require additional geotechnical investigation and seismic analysis. Refer to the approved AASHTO LRFD specifications and DM-4 for additional details.

Soils having an AASHTO Classification of A-7-5 or A-7-6 may fall in Site Class E criteria. While both classifications are generally rare in Pennsylvania, they can be found in abundance in certain locations. Soils having an AASHTO Classification of A-2-4, A-3 and A-4 may fall in Site Class F criteria. These include soils with high low to non-plastic silt contents and/or high fine sand contents. While A-3 classified soils are rare in Pennsylvania, isolated deposits are occasionally encountered. Soils with A-2-4 and A-4 classifications are conversely quite common in Pennsylvania, being found in high abundance. Materials with an A-4 classification account for approximately 40% of PA soils, while A-2-4 classified materials accounting for approximately 25%. Only A-2-4 materials of leaning towards a more uniform gradation having high silt and fine sand contents, are typically of concern for liquefaction.

6.14 PROBABILITY OF FAILURE FOR SLOPE STABILITY ANALYSIS

This section describes the relationship between factor of safety (FS) and probability of failure (p_f). Probability of failure (p_f) expresses how likely a failure is to occur. It can be expressed as a decimal fraction, percent, or odds (chance) that a failure will occur (e.g., 1 in 1000, etc.).

6.14.1 Factor of Safety vs. Probability of Failure

There are a variety of methods to determine p_f including mathematical formulation, point estimate methods, and Monte Carlo simulation used for geomechanical models. More recent studies have been conducted expressing p_f as a combination of key performance indicators including human factors, and quality of studies, design, construction, monitoring, and maintenance. Such key performance indicators for p_f are generally empirical or semi-empirical.

Both FS and p_f can be used to quantify the hazard a given slope exhibits relative to potential failure. The FS quantifies the state of equilibrium of a mass defined by a slope cross and a specific potential failure surface. The p_f quantifies the probability of failure for the same mass according to the selected engineering properties of the slope mass and the anticipated variability of those properties. For this discussion, both p_f and FS are based upon limit equilibrium analysis and are not risk based. Note there are other approaches and methods for determining p_f. These are beyond the scope of this discussion.

A FS is a measure of a slope’s ability to withstand the applied loads. Mathematically, the FS against slope failure is the ratio between resisting forces and driving forces. Some uncertainty will always exist concerning both the resisting forces and driving loads; however, there is typically more uncertainty with the available resisting capacity. For resisting capacity, the primary uncertainty involves the variability and predictability of the soil shear strength parameters (cohesion and internal friction angle). For driving loads there is uncertainty concerning soil unit weight, water table position and elevation, and externally applied loads (e.g., adjacent structures, etc.).

When performing slope stability analyses, it is common practice to account for uncertainties by reducing resisting forces and increasing driving loads to account for uncertainties in both, while also selecting a conservative target design FS. Probabilistic methods allow these uncertainties to be considered.

6.14.2 Probability Analysis and Recommendations

A probability analysis is unique to the slope design including the soil shear strength parameters. The variances assigned to those parameters are typically subjectively determined or estimated. The assigned variance values may or may not be appropriate; therefore, it may or may not accurately reflect the uncertainty in assigning the soil strength parameters used in both the limit equilibrium analysis and the probability analysis. There is no single or absolute answer to the probability analysis, but only answers reflecting the quality of the data assigned for the analysis. The same parameters and constraints that exist for the limit equilibrium analysis serve as the basis for the probability analysis.

Preparation of a conventional slope stability analysis model is dependent on how well the analysis model reflects reality. On the input side of a slope stability model, standard engineering practice is to select conservative soil shear strength parameters (cohesion and friction). This is done because of the uncertainty associated with these parameters. On the output side, a minimum FS (i.e., 1.3) is required to help ensure long-term stability and reduce risk of failure.

Both the p_f and FS practices substantially reduce the risk and probability of failure. Therefore, when following standard engineering practice (and for the large majority of situations), the observed rate of failure simply does not justify the additional time and resources required for a probability analysis. A condition of low p_f is already built into the procedures of standard engineering practice for the slope stability analysis. However, that specific level of risk cannot be quantified when employing a FS Analysis.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 7 – SETTLEMENT

CHAPTER 7 – TABLE OF CONTENTS

7. 1 INTRODUCTION

The potential for settlement must be considered during the design of all Department engineering projects. This chapter of this publication provides guidelines, recommendations and considerations for settlement analyses of structure foundations, embankments, and highway construction. As it is not possible to provide universal settlement analysis requirements to satisfy all Department projects, the requirements provided herein are appropriate for most Department projects. If project specific settlement requirements differ from those presented in this chapter, those requirements must be specified in the geotechnical scope of work. Any proposed changes to the settlement analysis requirements presented herein, must be approved by the Chief Geotechnical Engineer (CGE). To obtain approval, submit thorough documentation demonstrating the need and provide adequate supporting technical justification that the proposed change(s) is/are both required and appropriate for the specific site conditions and circumstances.

The analyses of foundation settlements are ultimately governed by Publication 15M, Design Manual 4. This chapter includes discussions related to foundation settlement and settlements associated with roadway embankments. In general, the discussions presented make reference to and concur with publications FHWA-NHI-06-088 and FHWA-NHI-06-089.

7.2 MECHANISM AND TYPES OF SOIL SETTLEMENT

Soils settle or compress when load is applied to the matrix of soil particles. For a very loose disturbed soil, the load could simply be the downward movement of surface waters. More typically, loads are the result of natural deposits of soil sediment over geologic time, or human-made deposits from construction activity (e.g., placement of fill materials, building structure bearing directly on soil, etc.).

When a load is applied to a soil mass, stresses increase within the soil mass and the individual particles of the matrix are pushed closer together. The load reduces the void space between soil particles by expulsion of air and/or water. The compression of the soil particles into a tighter arrangement results in a decrease in the volume of the soil mass. The decrease in volume is observed as settlement through downward movement or deformation at the ground surface. The total settlement, S, is the sum of the following three components:

$$S = S_e + S_c + S_s$$

where,

S_e = immediate or elastic settlement
S_c = primary consolidation or time dependent settlement
S_s = secondary compression settlement

Granular cohesionless soils undergo primarily immediate elastic settlement. Fine grained soils experience a combination of both immediate elastic settlement and time dependent consolidation settlement. Secondary compression settlement (sometimes referred to as creep or secondary consolidation) is long-term occurring in fine grained soils after the completion of consolidation settlement.

Depending upon both the nature of the load and the composition of the soil mass, the settlement may be uniform or irregular across the loaded area. A uniformly loaded area over a uniform and consistent soil mass, will tend to settle evenly across the loaded area. Non-uniform loads or loads on a soil mass of variable strength and/or layer thicknesses, can result in highly variable or differential settlement across the loaded area.

Generally, settlements affecting roadway features consist of immediate and long-term soil deformations. Long-term deformations can be separated into primary and secondary settlements based upon the mechanism involved. Immediate settlement is usually referred to as elastic settlement and is associated with higher permeability free draining granular soils. Long-term settlement is referred to as consolidation settlement and is associated with lower permeability fine-grained plastic soils at or near a saturated condition. Two distinct components in the analysis of consolidation settlement are magnitude (see following Section 7.3) and time rate (see Section 7.7).

7.2.1 Immediate (Elastic) Settlement

Immediate settlement is also referred to as elastic compression. It can be observed in dry soils, moist soils having deformation without a change in water content, or saturated free draining granular soils. The practice of referring to immediate settlement as elastic compression derives from the loaded soil exhibiting an increase in strain that is proportional to any increase in stress; however, soils are not truly elastic and do not retake their initial volumes and dimensions upon unloading (i.e., most permanent deformation remains). Equations that are derived from the theory of elasticity are typically used to compute estimates of immediate settlement. For structure foundations on stiff cohesive soils, reference DM-4 for equations to calculate immediate (elastic) settlement.

7.2.2 Primary Consolidation Settlement

Primary consolidation settlement occurs in nearly saturated to fully saturated, non-free draining, fine-grained soils as the void spaces in the soil mass are compressed resulting in a reduction in volume of the soil mass. As the voids are compressed, pore water is expelled from the mass. As the soil mass is loaded, low permeability of the soil mass impedes the expulsion of water. As water is not compressible, the rate of soil compression is dependent upon how fast the water can drain from the soil. Therefore, consolidation settlement is time dependent, with the rate of consolidation a function of soil permeability. Estimates of primary consolidation settlements must include consideration of not only the strength and deformation characteristics of the soil mass, but also its permeability. The soil permeability controls the rate at which pore water can be expelled from soil voids, also termed time rate of consolidation analysis, which is further covered in Section 7.7.

Cohesionless soils, which drain relatively fast, do not exhibit long-term consolidation. Also, cohesive soils that are dry or lack significant amounts of pore water do not generally experience significant long-term consolidation settlements. Consolidation settlement of low moisture content, unsaturated, plastic soil is generally small in comparison to the immediate settlements. Therefore, for saturated cohesionless soils with high non-plastic fines such as silts, and dry cohesive soils, the relatively small consolidation portion of the total settlement is frequently ignored for estimation purposes.

7.2.3 Secondary Compression Settlement

Secondary compression settlement (sometimes referred to as creep settlement) occurs after primary consolidation settlement has completed and is the result of reorientation of the soil particles and ongoing plastic deformation of the soil mass under constant loading. Organic soils and high plasticity clays are most affected by secondary compression. Micaceous soils are also prone to secondary compression settlement.

The mechanism of secondary compression occurs in the microscopic realm and is not yet clearly understood. Essentially, secondary compression is derived from the breakdown of bonds between individual clay particles. Secondary compression is treated separately from primary consolidation. Estimates of magnitude of the two are independent. Because the mechanism of secondary compression is not well-defined, secondary compression settlement calculations are taken to be only general approximations.

7.3 STRESS DISTRIBUTION

The ability to make suitable estimations of stresses in soils induced by external loadings is essential for realistic predictions of future settlements at roadway embankment and structure foundation locations. This section presents methods for computing and developing distributions of stresses beneath roadway embankments and structure foundations.

Likely the most commonly used basis for traditional computations of stresses in soils that are induced by external loadings is the theory of elasticity. Boussinesq originally developed the most frequently used formulas that employ this theory. Originally generated for point loads, the formulas have been enhanced to provide solutions for loaded areas (e.g., embankments, footings, etc.). The Boussinesq solutions are reliant on several simplifying assumptions. Primarily, the affected soil mass is assumed to be homogeneous and isotropic, and strains are proportional to stresses. Considering these assumptions, the elastic, theory-based solutions yield suitable approximations of stresses within the soils underlying the externally applied load. A variety of influence charts are presented below for use in estimating stresses under embankments and spread footings. Additional charts for other footing and loaded area shapes are provided in Design Manual, Part 4 (DM-4).

7.3.1 Embankment Foundations

The magnitudes of settlements and other deformations in embankment foundation soils are directly related to the magnitude of the pressure imposed by the embankments. The type of foundation settlement that occurs is a function of the composition and stress history of the embankment foundation soils. Immediate elastic deformation will occur in all soil types. Consolidation settlements are usually observed in saturated, fine-grained, cohesive soils. Methods for estimating elastic settlement magnitudes are discussed in Section 7.4. Similarly, a method for predicting consolidation settlement values is presented in Section 7.5.

Regardless of the type of soil and settlement being evaluated, the designer must make an estimate of the distribution of stresses in the embankment foundation that will be induced by loading from the embankment soil. The most versatile method is one presented in a paper titled "Influence Values for Vertical Stresses in a Semi-infinite Mass due to an Embankment Loading" It is known as the Osterberg method (Figure 7.3.2-1) and is available from the International Society for Soil Mechanics and Geotechnical Engineering website.

The method allows simple estimation of vertical stresses at any depth below an embankment and any location inside or outside the embankment footprint. The method uses a chart to determine the influence value from the loaded area to calculate the estimated vertical stress at the depth and desired location in the compressible foundation layer. Examples demonstrating the Osterberg method are presented in the Settlement – Worked Examples webpage.

7.3.2 Uniformly Loaded Strip and Square Footings

A method very similar to the Osterberg chart for embankments will be used for estimating the vertical stress under footings. Like the Osterberg chart, and most other methods for estimating vertical stresses under loaded areas, the Newmark chart (Figure 7.3.2-2) is based on the Boussinesq equation. Newmark performed an integration of the equation for rectangular loaded areas.

The Newmark method allows simple estimation of vertical stresses at any depth below a spread footing, and any location inside or outside the limits of the footing. The method uses a chart to determine the influence value from the loaded area to calculate the estimated vertical stress at the depth and desired location in the compressible foundation layer. The influence factor is for the corner of the footing; therefore, the principle of superposition will again be used for any location other than the corner of a footing. As with the Osterberg method, the Newmark method is simple but not intuitive without a series of short simple examples to illustrate its use. Examples demonstrating the Newmark method are presented in the Settlement – Worked Examples webpage.

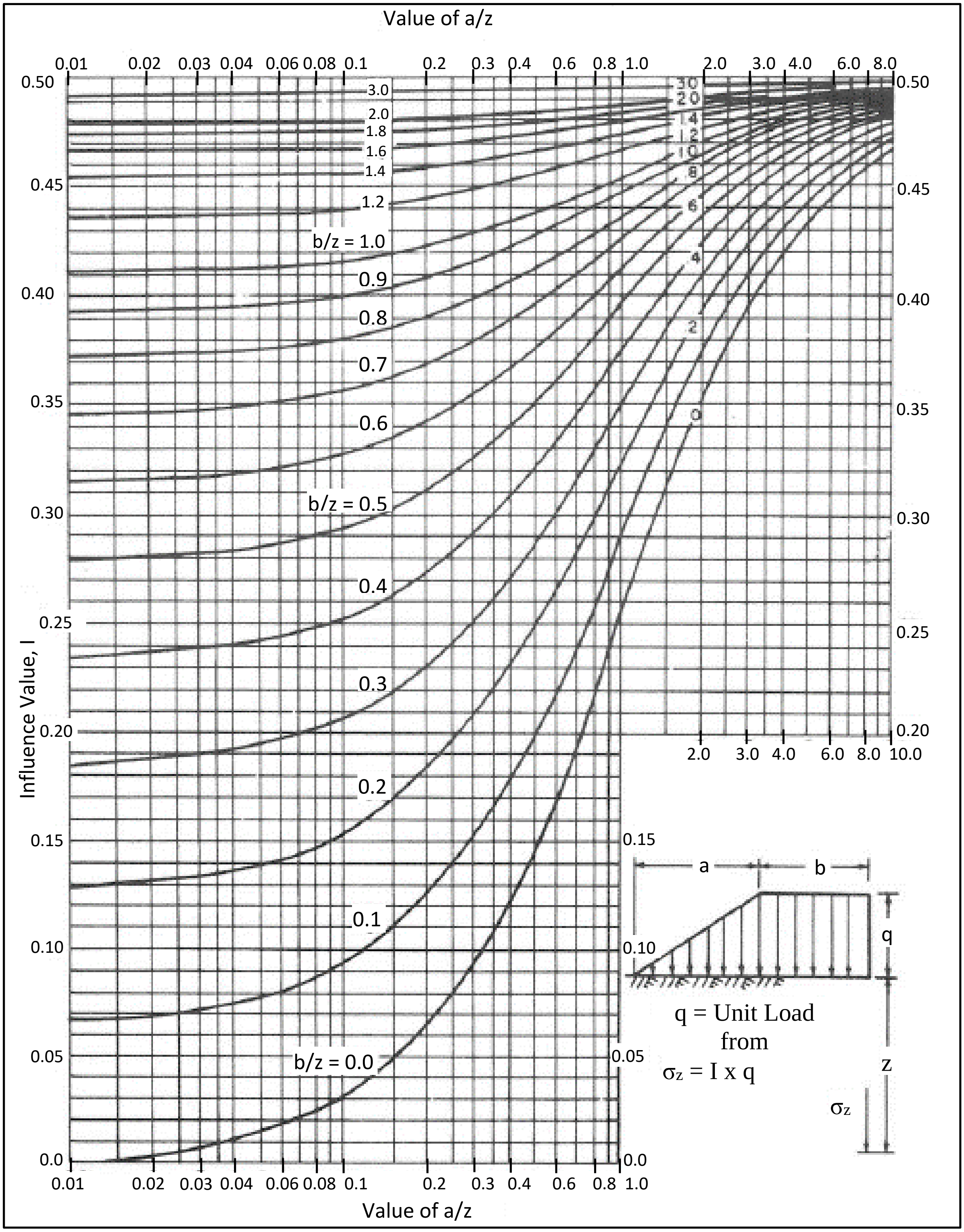


Figure 7.3.2-1 – Osterberg Influence Chart for Vertical Stress from Embankment Loading – Infinite Extent Boussinesq Case

Notes: 1. Referenced from NAVFAC DM 7.1 – 1982

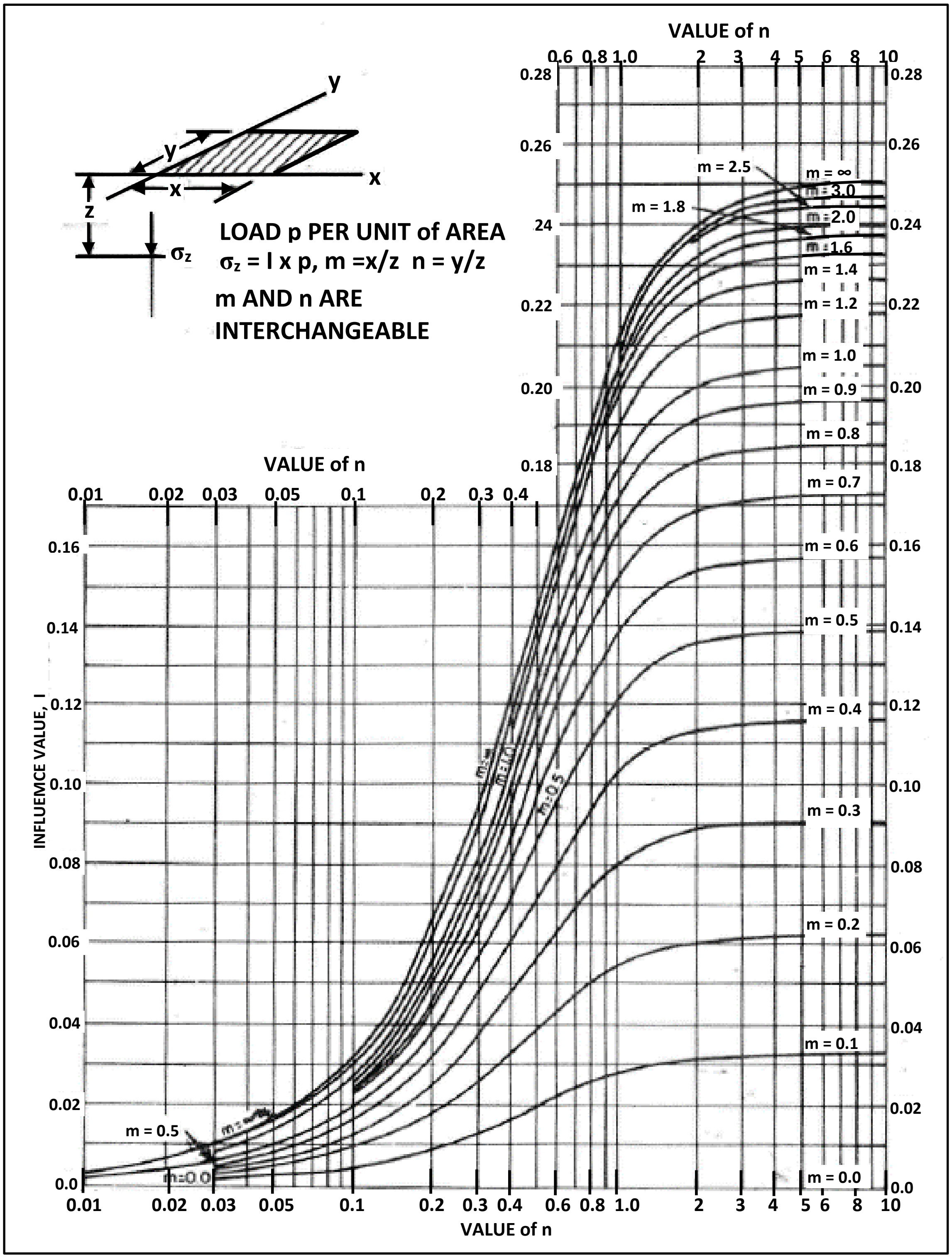


Figure 7.3.2-2 – Newmark Influence Chart for Vertical Stress Under Spread Footing

Notes: 1. Referenced from NAVFAC DM 7.1 – 1982

Figure 7.3.2-3 shows vertical stress contours for continuous and square footings using Boussinesq's theory. This diagram can provide a simple rapid estimate of vertical stresses useful for preliminary analyses only.

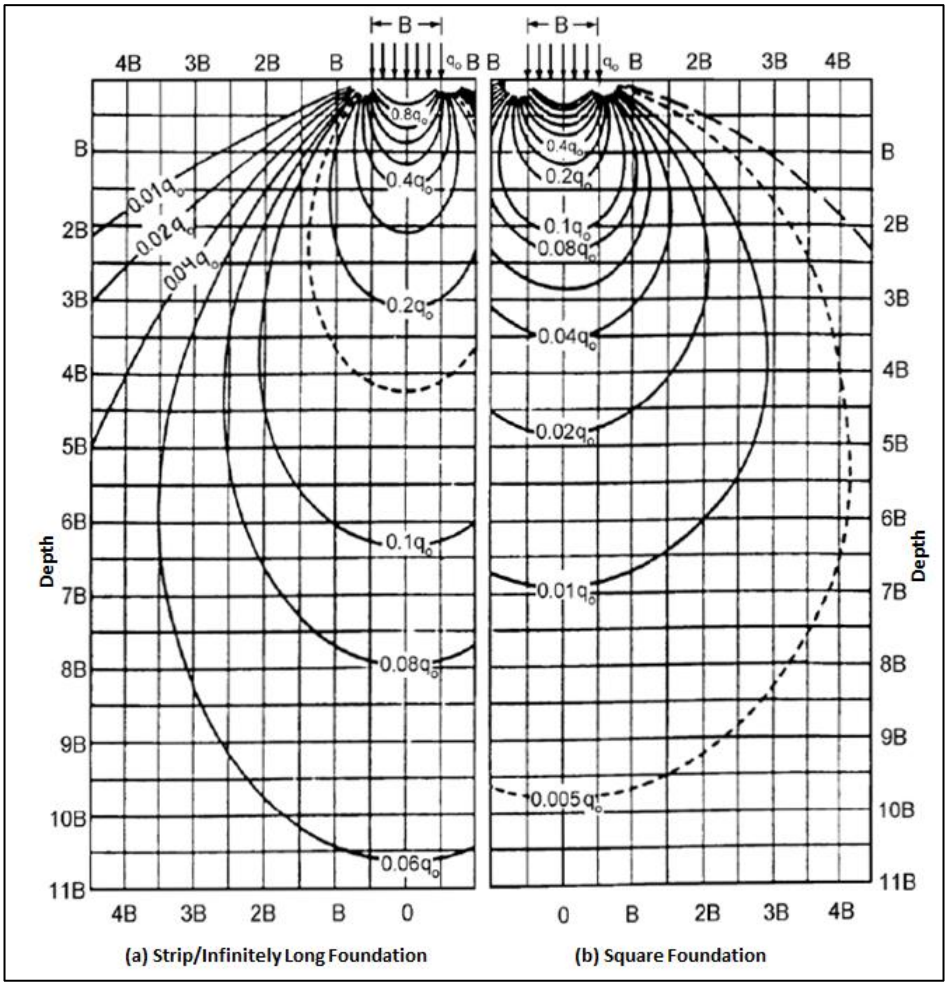


Figure 7.3.2-3 – Vertical Stress Contours (isobars)

Notes: 1. Based on Boussinesq's theory for continuous and square footings

2. Modified after Sowers, 1979; AASHTO, 2002 (FHWA, 2006).

7.3.3 Depth of Influence

An important step in calculating settlement from an applied load is determining the practical depth of influence of the load. A depth of twice the footing width or 2B is often used for preliminary analysis. While this may be acceptable in some situations, or strictly for preliminary purposes, it does not ensure that an adequate representation of settlement magnitude will be obtained. Relative to settlement, the depth of influence of an applied load is dependent upon several factors including the area and geometry of the load, the load magnitude, and the subsurface conditions (e.g., type of soil materials present, compressibility of the soil materials, depth to rock, etc.). Typically, the increase in vertical stress from the applied load should be 5% to 10% of the existing in-situ vertical stress. Note that this is not an influence value (I) of 5% to 10%, rather 5% to 10% of the initial vertical stress (i.e., the stress in the foundation soil before application of the new load). Examples demonstrating the determination of depth of influence based upon the above criteria are presented in the Settlement – Worked Examples webpage. Note that ABLRFD Manual v1.18.0.0, Section 3.4.1.3.2, states that the depth of soil considered for settlement is restricted to the lesser of the depth of the lower soil layer or 6B below the bottom of footing.

7.3.4 Preliminary Settlement Estimate – Approximate (1H:2V) Stress Distribution

A simplified 1H:2V approximation of stress distribution based upon the Boussinesq solutions has been employed by engineers for many years. This method of approximating vertical stress may only be applied for preliminary settlement analyses. The total contact pressure of the external loading is distributed over an area with the same aspect ratio as the actual loaded area, but with dimensions that increase with depth. The dimensions increase at the rate of one horizontal unit for each two units of depth, 1H:2V. This approximation is illustrated in Figure 7.3.4-1. At depth *z*, for a loaded area with dimensions *B x L* at the surface, the total contact load is distributed over an area of *(B+z) x (L+z)*. As drawn, the figure estimates the vertical stress increase to a depth of "B". This estimate of vertical stress increase would be used to estimate the settlement to a depth of "2B" (the average vertical stress increase to a depth of "2B" would be at the middle of the layer at a depth of "B"). "Part B" of the figure presents a comparison of the Boussinesq with the 1H:2V method at a depth of *B* below the contact surface.

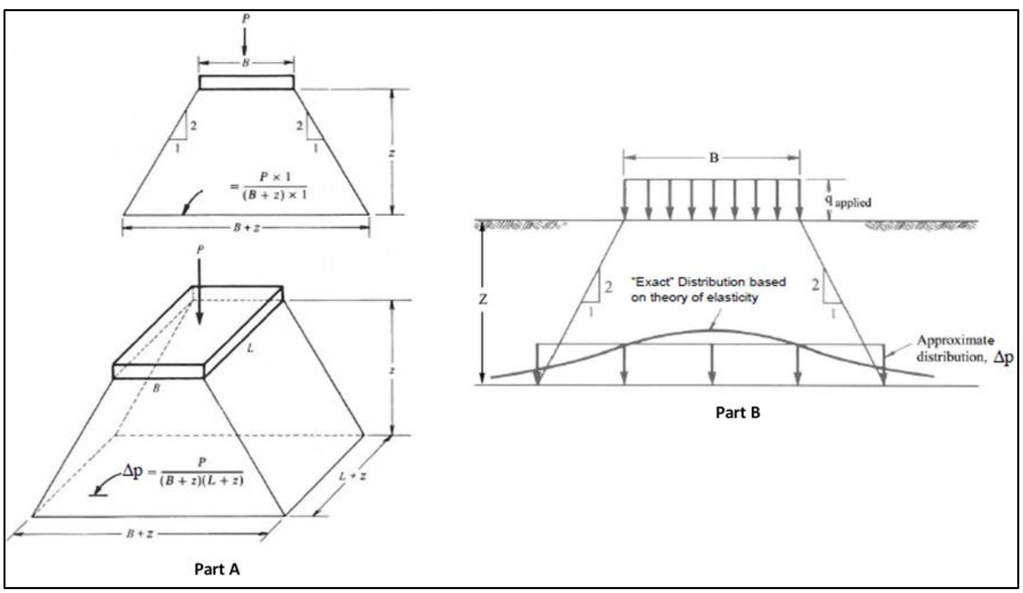


Figure 7.3.4-1 – Distribution of Vertical Stress by the 1H:2V Method

Notes: 1. After Perloff and Baron, 1976 (FHWA, 2006).

7.4 IMMEDIATE (ELASTIC) SETTLEMENT

Immediate settlements will occur in all soils, cohesive or cohesionless, as loads are applied to them. The settlements may continue with time after the application of a load and will be a function of how quickly the water can drain from the voids. Many methods for calculating estimations of immediate settlements have been developed and published. The following subsections present acceptable procedures for estimating settlements induced by embankment and structure foundation loadings.

7.4.1 Methods for Roadway Embankment Locations

7.4.1.1 Modified Hough Method

The method developed by Hough, and subsequently modified by AASHTO, provides an efficient and simple tool for estimating settlements below embankments. The modified Hough method is presented in FHWA-NHI-06-088, Article 7.4.1. In its current form, the method uses corrected SPT N-values ($N1_{60}$) to estimate a bearing capacity index (C'). The method was developed for normally consolidated cohesionless soils; therefore, it is not reliable for immediate settlement estimates in plastic soils. Also, the modified Hough method tends to yield estimated elastic settlement values that are somewhat conservative.

As presented in FHWA NHI-06-088, the steps required for the modified Hough method are summarized below.

1. Where appropriate, divide the layer being evaluated into sublayers not thicker than ten feet.

2. Determine the bearing capacity index (C') for each layer or sublayer by using Figure 7.4.1.1-1 with the appropriate $N1_{60}$ value and visual description of the soil.

3. Compute the estimated settlement for each layer or sublayer using the following equation and sum the incremental solutions. The sum of the incremental solutions is the estimated total immediate settlement of all affected layers and sublayers.

$$\Delta H = H\left(\frac{1}{C'}\right)\log_{10}\left(\frac{p_o + \Delta p}{p_o}\right)$$

where,

ΔH = settlement of layer or sublayer (ft)
H = thickness of layer of sublayer (ft)
C' = bearing capacity index (see Figure 7.4.1.1-1)
p_o = existing effective overburden pressure (psf) at the center (midpoint) of the layer or sublayer being evaluated. To avoid unrealistic settlement estimates, a minimum value of 200 psf is recommended for shallow surface deposits.
Δp = Proposed embankment pressure (psf) at the center (midpoint) of the layer or sublayer

The modified Hough method typically yields conservative settlement estimates that are satisfactory for embankment locations. For structure foundation locations, high estimates may be impractical for use in design. Shallow foundation settlements are discussed in Section 7.4.2. The requirements of DM-4 must be met for shallow foundations.

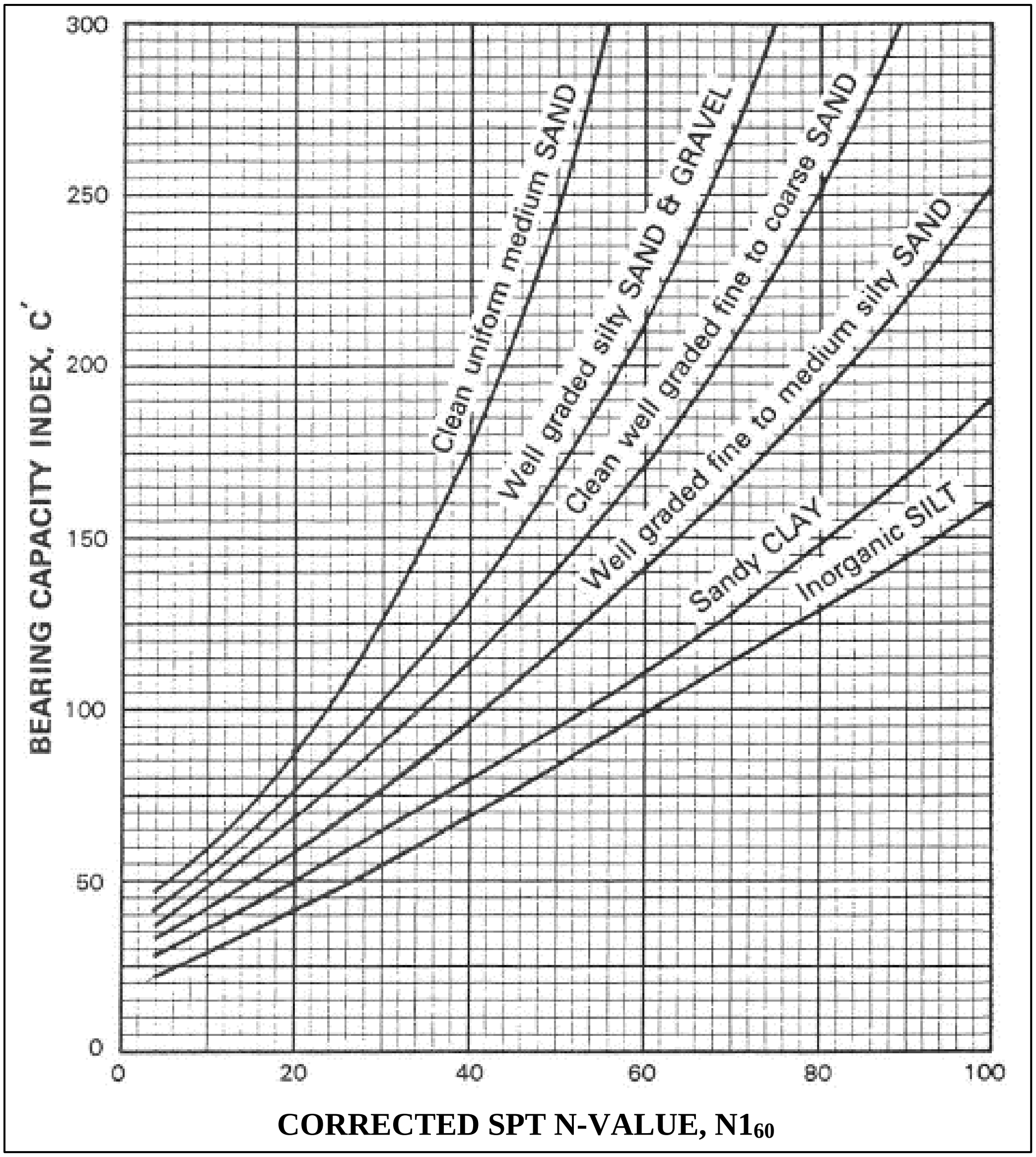


Figure 7.4.1.1-1 – Bearing Capacity Index (C') Values

Notes: 1. The "Inorganic Silt" curve must not be applied to soils that exhibit plasticity because N-values in such soil are unreliable.

7.4.1.2 Modified Schmertmann Method for Embankments

The modified Schmertmann method provides a more rigorous process for estimation of immediate settlements that can yield more accurate, realistic results. The method can be used for a more advanced analysis of cohesionless and cohesive soils. Additionally, because the modified Schmertmann method delivers more suitable estimates of immediate settlements across a wide range of soil types, it is specifically recommended for use at structure foundation locations as presented in the following subsection.

7.4.2 Shallow Foundations - Modified Schmertmann Method

The requirements of DM-4 must be met for shallow foundations. However, the modified Schmertmann method can be used for verification of the settlement results as it is an effective procedure for determining estimates of immediate settlements in various soil types. The modified Schmertmann method accounts for the effects of lateral strain induced by foundation loads on the vertical settlements and incorporates a correction factor to account for footing embedment. Also, modified Schmertmann's method can be used to estimate creep deformation settlement values at various times after load application. The modified Schmertmann method is presented in FHWA-NHI-06-089, Article 8.5.1.

The general equation used to compute estimated settlement using the modified Schmertmann method is detailed below. FHWA NHI-06-089, Article 8.5.1, also provides a detailed, worked example problem that illustrates the modified Schmertmann method.

Modified Schmertmann Method for Calculation of Immediate (Elastic) Settlement:

$$S_i = C_1 C_2 \Delta p \sum_{i=1}^{n} \Delta H_i$$

where,

$$\Delta H_i = H_c \left(\frac{I_z}{XE}\right)$$

and:

S_e = immediate settlement (ft)
H_c = thickness of soil layer being analyzed (ft)
I_z = strain influence factor (see Figure 7.4.2-1). This factor is a function of depth and is determined from the strain influence diagram.
n = number of soil layers in the zone of strain influence determined from the strain influence diagram
Δp = proposed net applied stress at the foundation depth (see Figure 7.4.2-2)
E = elastic modulus of the soil layer being analyzed (refer to AASHTO Table C10.4.6.3-1)
X = modification factor for the value of the elastic modulus

If the value of E is based on correlations with $N1_{60}$ or static cone testing, then:
X = 1.25 for the axisymmetric case ($L_f / B_f = 1$)

X = 1.75 for the plane strain case ($L_f / B_f \geq 1$)
Interpolate for footings with $10 \geq L_f / B_f \geq 1$

If the value of E is based on typical reference values, then:
X = 1.0

C_1 = Correction factor for strain relief due to footing embedment.

$$C_1 = 1 - 0.5\left(\frac{p_o}{\Delta p}\right) \geq 0.5$$

where,

p_o = existing effective overburden pressure at the footing elevation (see Figure 7.4.2-2)

C_2 = Correction factor for time-dependent creep deformation for t years after construction.

$$C_2 = 1 + .02\log_{10}\left(\frac{t_{(years)}}{0.1}\right)$$

where,

t = desired time of creep deformation in years ($t = 0.1$ for computation of immediate settlement)

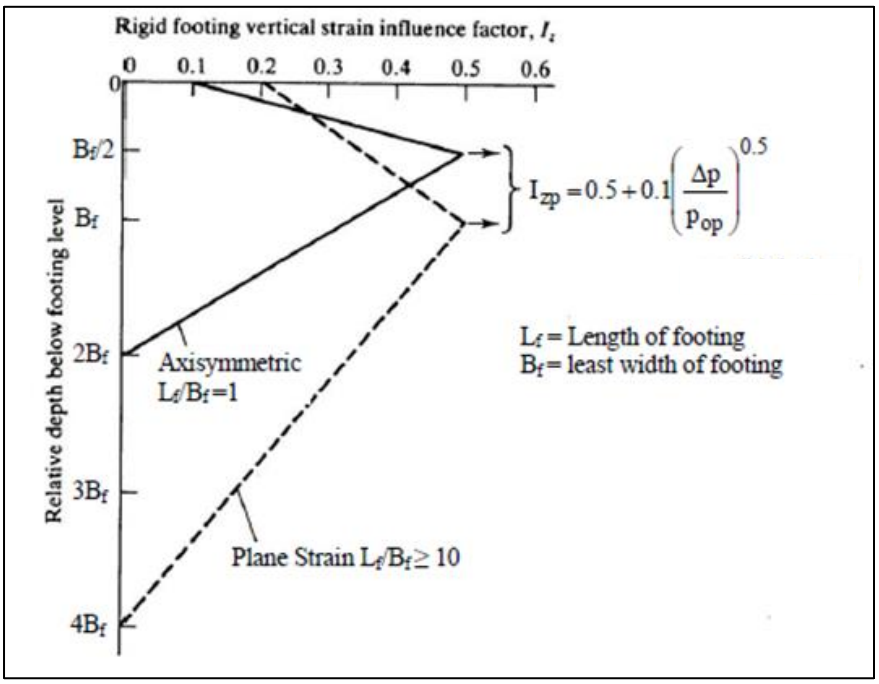


Figure 7.4.2-1 – Vertical Strain Influence Factor Diagram

Notes: 1. Adapted from Reference: FHWA, 2006

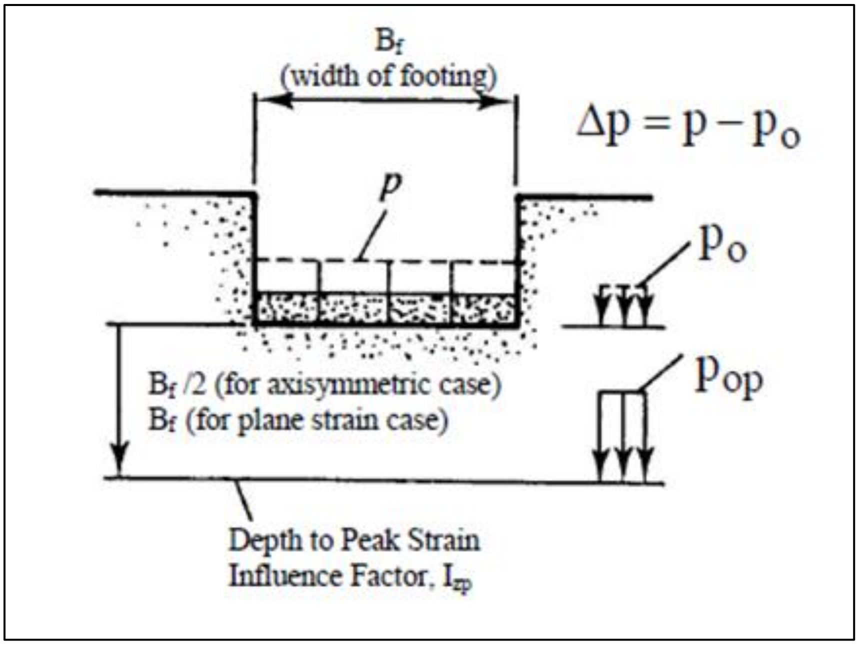


Figure 7.4.2-2 – Pressure Terms Definitions

Notes: 1. Adapted from: FHWA, 2006

7.5 PRIMARY CONSOLIDATION (LONG-TERM) SETTLEMENT

When saturated, fine-grained soils are exposed to significant additional loadings, long-term consolidation settlements will likely develop. While consolidation settlements are generally associated with soils that are fully saturated, it is entirely possible that consolidation settlements may also develop in soils that have high degrees of saturation. This situation can occur as the load itself facilitates reduction of the void ratio of the soil during immediate settlement, thereby effectively increasing the degree of saturation. When this occurs, the immediate settlement can lead directly to the onset of long-term, primary consolidation settlement. This section addresses methods for computing estimates of the magnitude of long-term settlements due to the consolidation process.

7.5.1 Laboratory One-Dimensional Consolidation Testing

One of the most important and commonly used approaches in estimating consolidation settlements is the one-dimensional consolidation test. The following subsections describe the standard method of interpreting and using the results of the test. Figure 7.5.1-1 shows a typical plot from one-dimensional consolidation test results, depicting a deformation vs log time curve. As can be observed in this figure, there are several stages to consolidation. If the soil mass has experienced a higher stress state in the past that is greater than the current overburden stress before loading, then it will undergo a recompression phase (i.e., Stage I: Initial Compression). A soil mass will undergo some level of elastic rebound if the load is removed. When the load being applied reaches an equivalent level it has been subjected to in the past, the mass will experience recompression.

Once the load is enough to exceed any historical or preload pressures, virgin compression initiates (i.e., Stage II: Primary Consolidation). Virgin compression is compression from load and overburden stress the soil mass has not experienced in the materials past stress history. Finally, after primary consolidation is complete, some soil materials will experience secondary consolidation or secondary compression (i.e., Stage III: Secondary Consolidation). Secondary compression occurs at a very slow rate and is not well understood. Secondary compression will be covered in more detail in Section 7.6. Whether significant secondary compression occurs depends upon the nature of the clay in the soil mass or if the material has a high organic content.

To predict the amount of consolidation in saturated, fine-grained, plastic, and organic soils, a consolidation test is performed on a specimen obtained from an undisturbed sample of the soil layer(s) that is considered to have the potential to exhibit time dependent settlement. Obtain an undisturbed soil sample in the field with a Shelby tube sampler. The oedometer or one-dimensional consolidometer is the primary laboratory equipment used to evaluate consolidation and settlement potential of fine-grained soils. AASHTO T216, Standard Method of Test for One-Dimensional Consolidation Properties of Soils, is the standard procedure for conducting consolidation testing of soils.

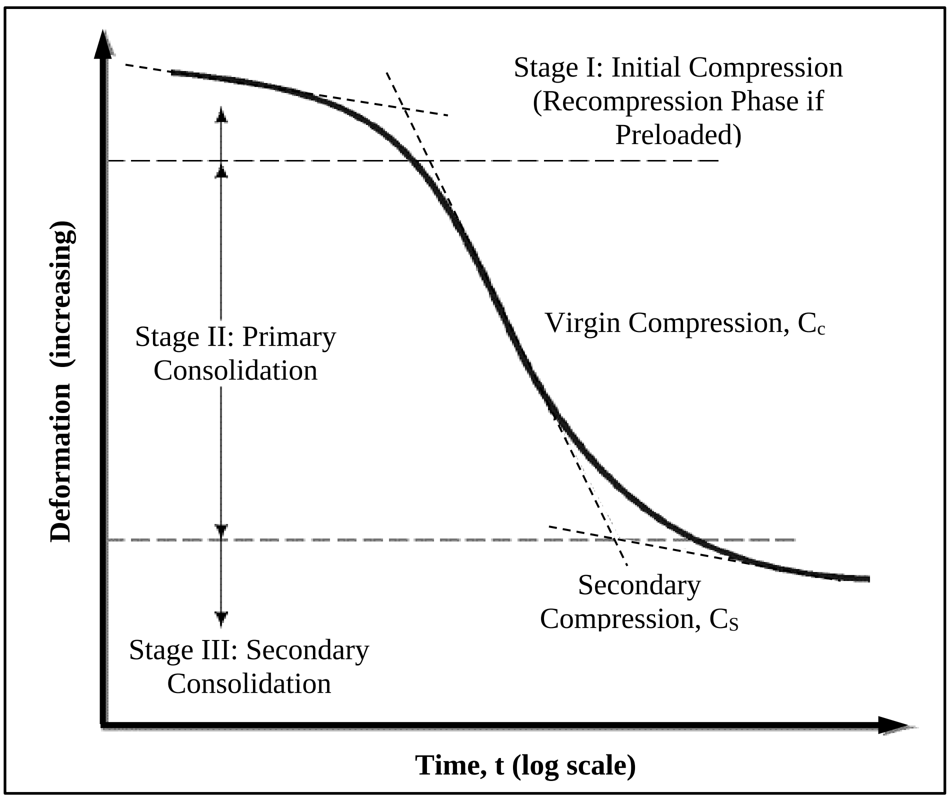


Figure 7.5.1-1 – Consolidation (Deformation vs Log Time) Curve

Notes: 1. Adapted from: Das FGE, 2005

Figure 7.5.1-2 displays a schematic of a consolidation test sample chamber. Common load frames include weighted lever arms and automated pneumatic or electronic equipment. As shown in this figure, the undisturbed sample is placed in the sample mold and then saturated with water. The sample is kept saturated for the entire length of the test. Porous stones are placed at the bottom of the mold and on top of the sample. This sample configuration models a two-way drainage condition representing a compressible soil layer with material layers above and below having permeabilities sufficiently greater than the test specimen. This configuration allows drainage of water from the test sample limited only by its own permeability. If the layer being modelled is underlain by rock or another impervious material, this would be considered a one-way drainage system, and the bottom porous stone would be replaced by an impervious insert to more accurately model field conditions.

The load is applied vertically to the sample. The load magnitude is calculated to create a pressure on the sample following the load schedule specified by the test method. Loads (and, therefore, pressures) are typically doubled for each subsequent load cycle, as further discussed in Chapter 4, Section 4.8.4.11. When a load is applied, readings are taken at prescribed intervals to track and record the consolidation process. These readings will allow plotting of the "e-log p" curve discussed in greater detail below. The consolidation test must be conducted so that enough time is allowed for the applied pressure increment to be transferred from the pore water to the soil structure. The time for the transfer of applied stress from pore water to the soil structure is a function of the soil permeability. In other words, the excess water needs time to drain out of the soil for the consolidation process to take place. During the drainage process, volume change occurs that models the primary consolidation settlement.

The time it takes for the transfer of stress to occur is the basis for the process being called "consolidation" and not "compression." The resulting effective stress corresponding to the applied pressure is plotted relative to the void ratio. The resulting "consolidation curve" allows an evaluation of the preconsolidation pressure and values for other parameters pertaining to the consolidation characteristics of the soil sample.

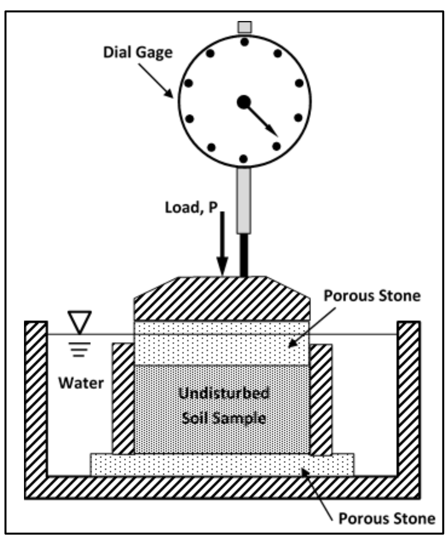


Figure 7.5.1-2 – Schematic of a One-Dimensional Consolidation Test Sample Chamber

Select the duration of each load increment to ensure that the sample is approximately 100% consolidated before application of the next load increment allowing the sample adequate time to drain so that all excess pore pressures are transferred to the soil structure. Twenty-four-hour load durations are typically applied; however, longer intervals may be necessary for some highly plastic materials.

Plots of void ratio versus effective pressure on arithmetic and logarithmic scales are shown in Figure 7.5.1-3. The semi log plot is more widely used in practice and will be used in subsequent sections of this publication. The consolidation curve on the void ratio versus semi log pressure plot is commonly referred to as the “e-log p” relationship. As shown on this figure, the slope of the loading portion of the “e-log p” curve is called the compression index, which is denoted by the symbol C_c. The slope of the re-load portion of the “e-log p” curve is called the re-compression index, which is denoted by the symbol C_r.

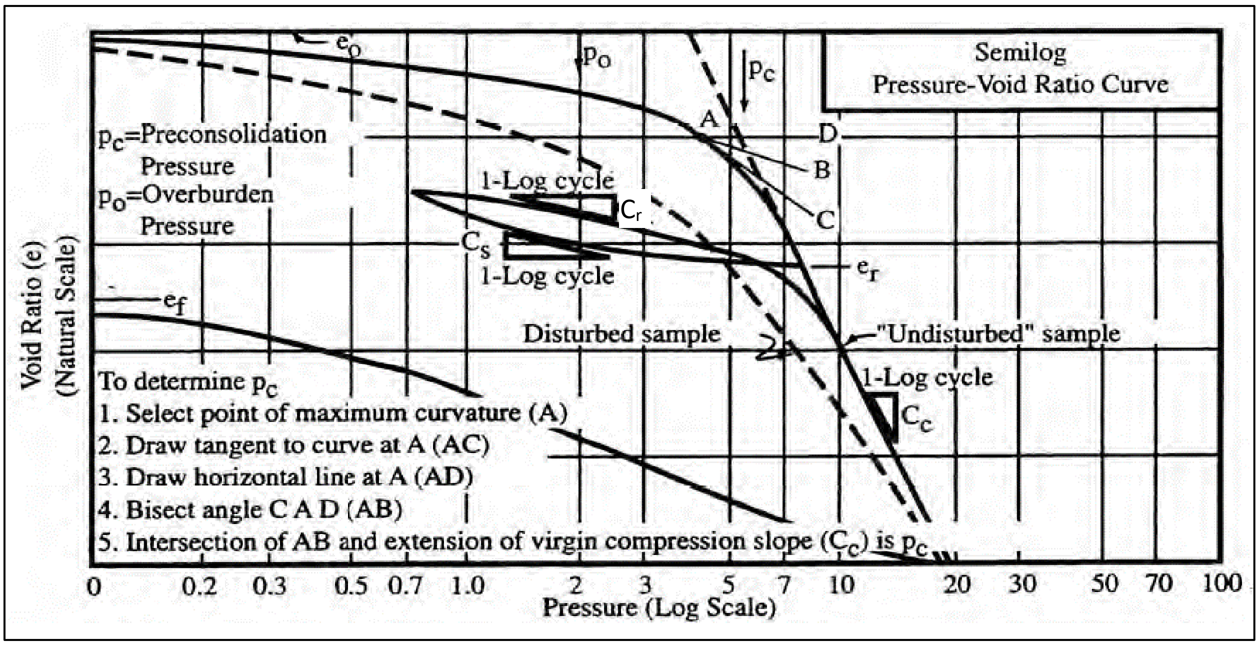


Figure 7.5.1-3 – Example of “e-log p” Curve

The value of the initial void ratio (e_o) is important because it defines the amount of void space at the start of the loading. It is critical that the value for initial void ratio be accurate. It is the initial void space that will be reduced as the water is squeezed out of the voids with time. The initial void ratio is a key parameter used in settlement computations to determine the magnitude of settlement.

Disturbance of soil samples occurring during collection of samples can result in a distorted consolidation curve. Typical sampling disturbances impact the portion of the curve that defines the transition from recompression to primary consolidation. The transition can appear to occur at a lower maximum past vertical pressure (p_c) than what would be observed for a completely undisturbed sample. As a result, correction of the “e-log p” curve is required. A good method for making the corrections to the consolidation curve is presented in Figure 7.5.1-4(a) for normally consolidated soils and Figure 7.5.1-4(b) for overconsolidated soils. A step-by-step procedure describing the correction is presented in Table 7.5.1-1.

Table 7.5.1-1 – Reconstruction of Field Virgin Consolidation Curve

Step	Description
	a. Normally Consolidated Soil (Figure 7.5.1-4(a))
1	By eye, determine point B at the point of minimum radius of curvature (maximum curvature) of the laboratory consolidation curve.
2	Plot point C by the Casagrande construction procedure: (1) Draw a horizontal line through point B; (2) Draw a line tangent to the consolidation curve at point B; (3) Draw the bisector between the horizontal and tangent lines; and (4) Draw a line tangent to the "virgin" portion of the laboratory consolidation curve. Point C is the intersection of the tangent to the virgin portion of the laboratory curve with the bisector and indicates the maximum preconsolidation (past) pressure, p_c.
3	Plot point E at the intersection of a horizontal line through e_o and the vertical extension of point C that corresponds to p_c as found from Step 2. The value of e_o is given as the initial void ratio before testing in the consolidometer.
4	Plot point D on the laboratory virgin consolidation curve at a void ratio ($e = 0.42e_o$). Extend the laboratory virgin consolidation curve to that void ratio if necessary.[2]
5	The field virgin consolidation curve is the straight line determined by points E and D.
6	The field compression index, C_c, is the slope of line ED.
	b. Overconsolidated Soil (Figure 7.5.1-4(b))
1	Plot point B at the intersection of a horizontal line through the given e_o and the vertical line representing the initial estimated in-situ effective overburden pressure, p_o.
2	Draw a line through point B parallel to the mean slope, C_r, of the rebound laboratory curve.[3]
3	Plot point D by using Step 2 above for normally consolidated soil.
4	Plot point F by extending a vertical line through point D up through the intersection of the line of slope C_r extending through point B.
5	Plot point E on the laboratory virgin consolidation curve at a void ratio of $e = 0.42e_o$.
6	The field virgin consolidation curve is the straight line through points F and E. The field reload curve is the straight line between points B and F.
7	The field compression index, C_c, is the slope of line FE

Notes: 1. Modified from USACE, 1994 per FHWA, 2006

2. On the basis of extensive laboratory testing, Schmertmann found that the laboratory curve intersects the field virgin curve at approximately $e = 0.42e_o$.

3. C_r = field recompression index

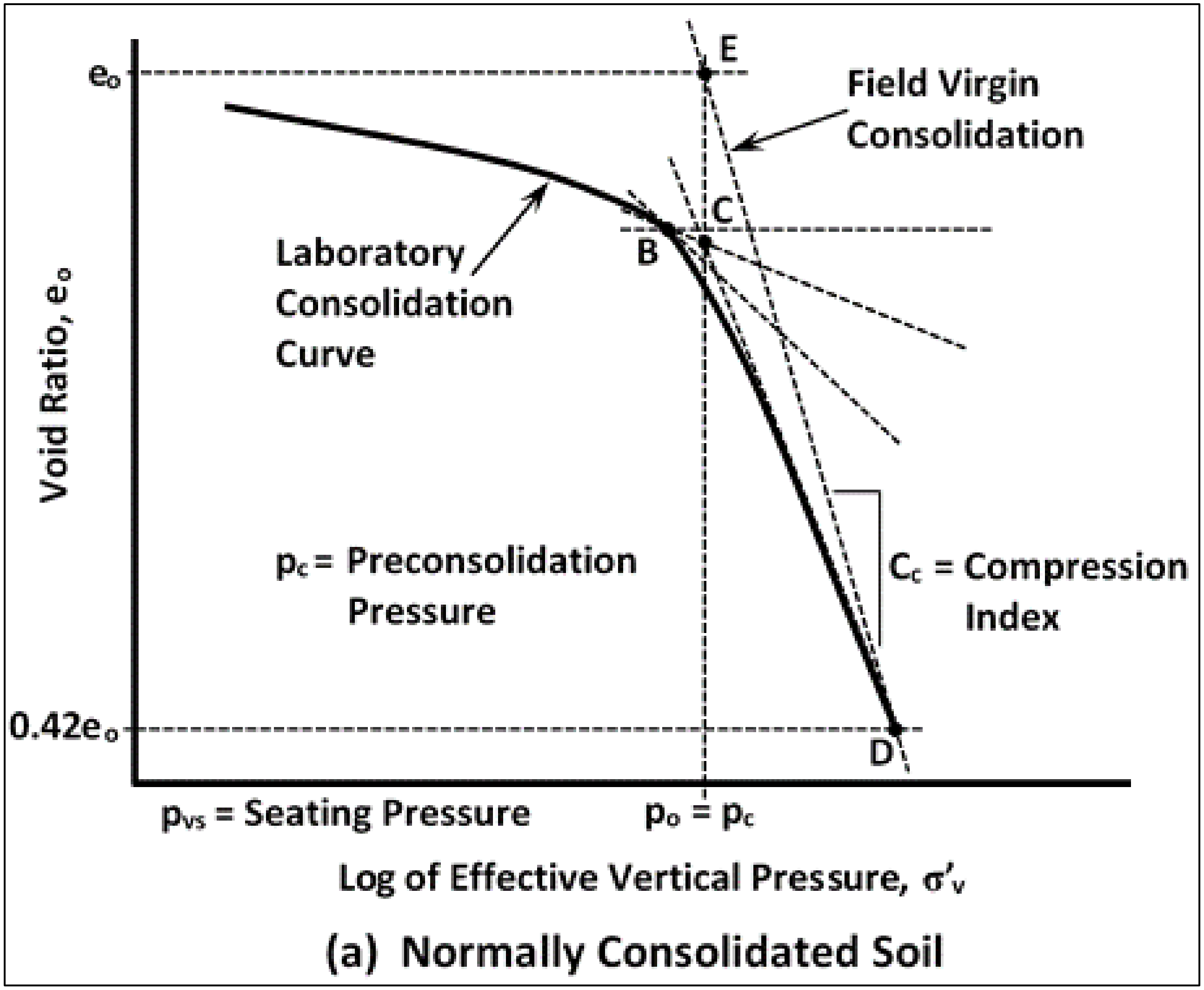


Figure 7.5.1-4(a) – Construction of Field Virgin Consolidation Relationships

Notes: 1. Adapted from: USACE, 1994 per FHWA, 2006

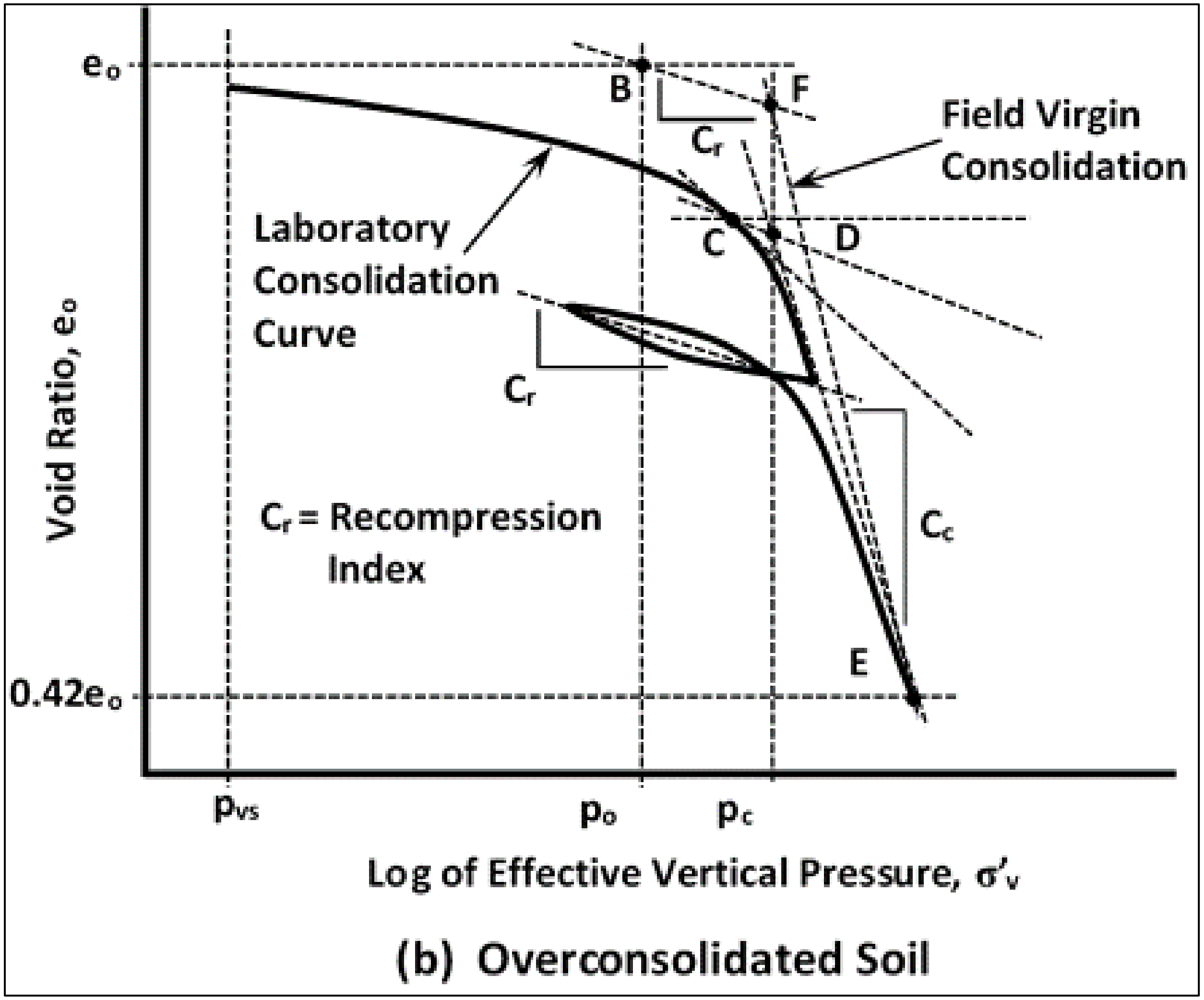


Figure 7.5.1-4(b) – Construction of Field Virgin Consolidation Relationships

Notes: 1. Adapted from: USACE, 1994 per FHWA, 2006

Sample size is a very significant limitation of one-dimensional consolidation testing. The height of the sample for a standard laboratory consolidation test is one inch. Natural soil deposits are not manufactured, but rather accumulate by natural processes whose mechanisms, depositional environments and conditions, and material sources, vary over time. Even during a specific depositional event in time, conditions can vary laterally quite significantly. Therefore, spatial variability and inconsistency of soil deposits is common and must always be considered during settlement analysis. The Engineer should consider collecting soil samples at different locations and depths and conduct testing to properly assess representative conditions and properties. Consolidation testing at regular intervals of depth will also aid in developing a representative stress profile (i.e., total and effective stresses as well as preconsolidation pressures profile). This approach can reduce the risk that could result in higher costs due to a completely inadequate or conservative design and treatment plan (e.g., cost overruns, construction delays, costly construction claims for delays, additional work, etc.). The cost of obtaining and testing a reasonable/greater number of soil specimens will be far less than the unforeseen costs of an inadequate or excessively conservative design and treatment plan.

In addition to one-dimensional consolidation testing with controlled stress according to AASHTO D2435, a Constant Rate of Strain (CRS) test (ASTM D4186) consolidation device could be used if consolidation parameters are needed quickly, or if there is a need to verify results for the coefficient of consolidation or hydraulic conductivity from consolidometer testing. Results of the CRS consolidation test may not be comparable to consolidometer test results where there is variation of soil stiffness with age. That is, where the days required for the one-dimensional consolidometer testing results in an increase of soil stiffness with age. The CRS test can be used to assess compressive behavior of cohesionless soils, but will not provide a measure of consolidation or hydraulic conductivity. The CRS test is not to be used for partially saturated soils. If the Constant Rate of Strain test is desired, please confirm testing availability with specific labs before specifying use of test method. A more in-depth discussion relative to CRS testing is currently outside the scope of this guidance chapter.

7.5.2 Computation of Primary Consolidation Settlements

The first step in the calculation of estimated long-term consolidation settlements is the determination of the stress history of the soils being evaluated. The definitions stated in terms of the Overconsolidation Ratio (OCR) are summarized below:

- OCR = 1 - the soil is “normally consolidated” under the existing load (i.e., the clay has fully consolidated under the existing load ($p_c = p_o$)).
- OCR > 1 - the soil is “overconsolidated” under the existing load (i.e., the clay has consolidated under a past load greater than the load that currently exists ($p_c > p_o$)).
- OCR < 1 - the soil is “underconsolidated” under the existing load (i.e., consolidation under the existing load is still occurring and will continue to occur under that load until primary consolidation is complete, even if no additional load is applied ($p_c < p_o$)). An OCR < 1 may occur for a limited range of conditions such as recent lowering of the groundwater table or possibly due to sample disturbance.

7.5.2.1 Normally Consolidated Soils

Estimated primary consolidation settlements induced by structure foundation or roadway embankment loadings in normally consolidated soils (p_c = p_o) are computed using the following equation:

$$S_c = \sum_{i}^{n} \frac{C_c}{1 + e_o} H_o \log_{10}\left(\frac{p_f}{p_o}\right)$$

where,

S_c = total consolidation settlement
n = number of layers into which the consolidating layer is divided
C_c = compression index
e_o = initial void ratio
H_o = layer thickness
p_o = initial effective vertical stress at the center of layer n
p_f = $p_o + \Delta p$ = final effective vertical stress at the center of layer n
Δp = change in vertical stress at center of layer n from additional applied load

The sum of the incremental settlements of the n individual layers is the total consolidation settlement.

As observed in Figure 7.5.2.1-1, the compression index (C_c) is the slope of the straight-line portion of the "e-log p" curve, which represents virgin compression (i.e., settlements from loads in excess of existing or past loads, p_o or p_c). The value of $\Delta p = p_f - p_o$. For a normally consolidated soil, Δp is simply the increase in pressure from the applied load at the mid-point of the layer, as determined according to Section 7.3.

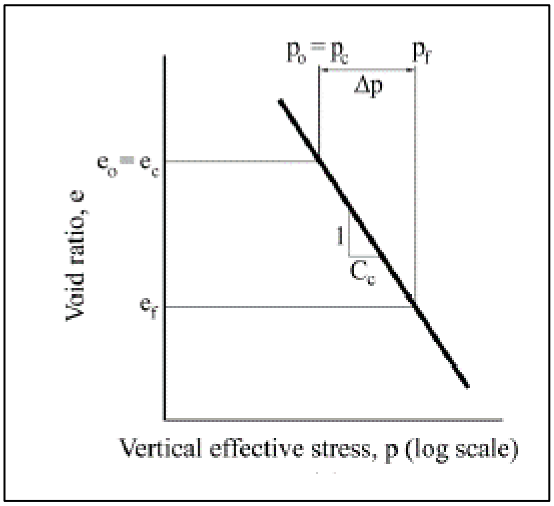


Figure 7.5.2.1-1 – Compression Index (Cc)

Notes: 1. C_c = Slope of straight-line portion of "e-log p" curve adapted from FHWA, 2006.

7.5.2.2 Overconsolidated (Preconsolidated) Soils

For overconsolidated soils ($p_c > p_o$), primary consolidation settlements induced by structure foundation or roadway embankment loadings are estimated using the following equation:

$$S_c = \sum_{1}^{n} \frac{H_0}{1+e_o}\left(C_r \log_{10}\frac{p_c}{p_o} + C_c \log_{10}\frac{p_f}{p_c}\right)$$

where,

S_c = total consolidation settlement
n = number of layers into which the consolidating layer is divided
C_c = compression index
C_r = recompression index
e_o = initial void ratio
H_o = layer thickness
p_c = preconsolidation pressure (i.e., maximum past effective stress)
p_o = initial effective vertical stress at the center of layer n
p_f = $p_o + \Delta p$ = final effective vertical stress at the center of layer n
Δp = change in vertical stress at center of layer n from additional applied load

The sum of the incremental settlements of the n individual layers is the total consolidation settlement. The recompression index (C_r) is the slope of the straight-line portion of the "e-log p" curve before the preconsolidation pressure (p_c) is reached. This can be seen in Figure 7.5.2.2-1 as the upper portion of the "e-log p" curve. This portion of the curve represents the recompression of an overly consolidated soil before the load exceeds the preconsolidation pressure (i.e., before virgin compression). As with a normally consolidated soil, the value of $\Delta p = p_f - p_o$; however, p_o is now located in the recompression portion of the "e-log p" curve.

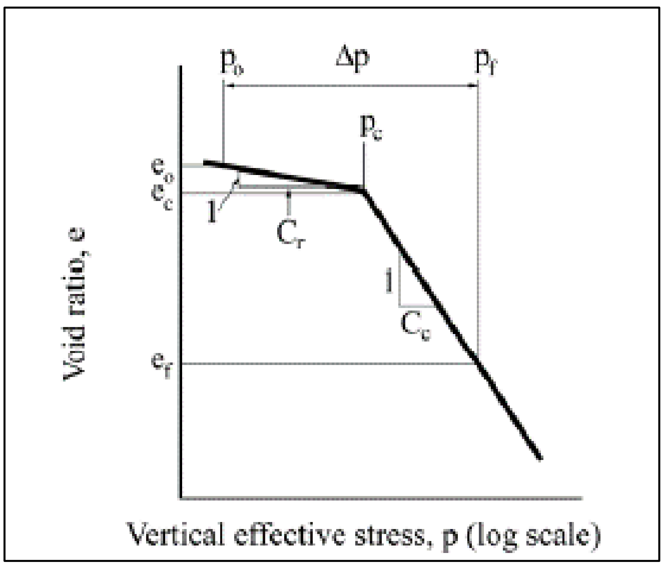


Figure 7.5.2.2-1 – Recompression Index (C_r)

Notes: 1. Adapted from FHWA, 2006.

Because the soil has been overconsolidated by a past stress that is greater than its current level, the existing stress state is located on the flat, recompression part of the consolidation curve. Therefore, the settlement of layer *n* is computed by adding the consolidation along the recompression and primary consolidation portions of the curve. Some, or possibly all, of the load applied goes into recompression of the soil layer, until the historical stress level (p_c) has been exceeded. Overconsolidation of soils can be the result of various situations that may have existed in the past, but do not exist in the present. These include the past existence of glaciers that have receded, erosion of past overlying soils, excavation of previous fills, removal of previous structures, development of negative pore pressures in fine-grained soils from desiccation and soil suction, etc. Although the Casagrande approach is recommended here and is current practice by the Department to determine the preconsolidation pressure, other methods can be used to verify results such as the Dissipated Strain Energy Method.

7.5.2.3 Underconsolidated Soils

Underconsolidated soils ($p_c < p_o$) can also experience long-term, consolidation settlements due to structure foundation or roadway embankment loads. The primary consolidation settlement of underconsolidated soil is estimated using the following equation:

$$S_c = \sum_{1}^{n} \frac{H_0}{1 + e_o}\left(C_c \log_{10} \frac{p_o}{p_c} + C_c \log_{10} \frac{p_f}{p_o}\right)$$

where,

S_c = total consolidation settlement
n = number of layers into which the consolidating layer is divided
C_c = compression index
e_o = initial void ratio
H_o = layer thickness
p_c = preconsolidation pressure (i.e., maximum past effective stress)
p_o = initial effective vertical stress at the center of layer *n* (for underconsolidated soils, $p_o > p_c$)
p_f = $p_c + \Delta p$ = final effective vertical stress at the center of layer *n* (Use p_c instead of p_o since primary consolidation is not complete for underconsolidated soils)
Δp = change in vertical stress at center of layer *n* from additional applied load [$\Delta p = (p_o - p_c) + (p_f - p_o) = (p_f - p_c)$]

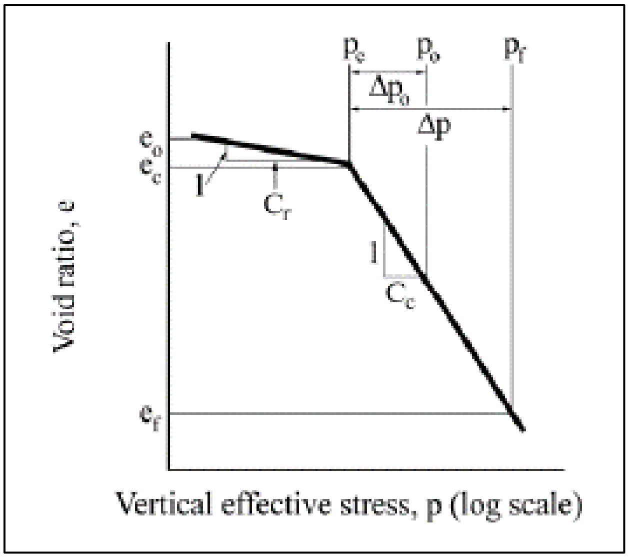


Figure 7.5.2.3-1 – Typical Consolidation Curve for an Underconsolidated Soil

Notes: 1. Adapted from FHWA, 2006.

2. $\Delta p_o = p_o - p_c$

The sum of the incremental settlements of the *n* individual layers is the total consolidation settlement. Underconsolidated soils have not completed primary consolidation for the current stress state. Therefore, for an accurate estimate of future consolidation settlement, the settlement associated with the existing loading must be considered in conjunction with the settlement expected because of the additional, new loading. Graphically, po is plotted neither along the recompression curve, nor at the beginning of the virgin compression curve, but somewhere along the virgin compression curve as shown in Figure 7.5.2.3-1. The value of $\Delta p = p_f - p_c$ for underconsolidated soils.

7.6 SECONDARY COMPRESSION

When organic soils and high plasticity clays are exposed to additional loads, long-term secondary compression settlement may develop. High plastic soils would include AASHTO classification of A-7-6 and Unified soil classifications of CH or OH (refer to Publication 222, Appendix D). This section presents a method for computing an estimate of long-term secondary compression that occurs after primary consolidation is complete. Secondary compression is observed in the settlement-log time plot when the sample consolidates beyond 100% of primary consolidation (i.e., beyond t_{100}) as shown below in Figure 7.6.1-1.

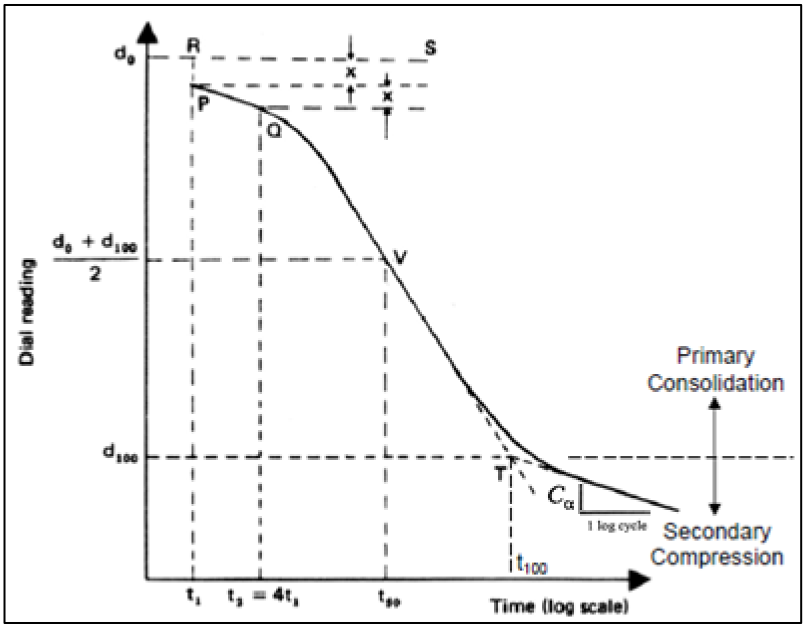


Figure 7.6.1-1 – Settlement-Log Time Plot

Notes: 1. Adapted from FHWA, 2006.

The magnitude is estimated as:

$$S_s = \frac{C_\alpha}{1 + e_o} H_c \log_{10}\left(\frac{t_2}{t_1}\right)$$

where,

- S_s = estimate magnitude of secondary consolidation
- t_1 = time when approximately 90% of primary compression has occurred for the actual clay layer being considered
- t_2 = the service life of the structure or any other time of interest of significance
- C_α = coefficient of secondary compression
- e_o = initial void ratio
- H_c = thickness of the compressible layer

Secondary compression is treated separately from primary consolidation. Estimates of magnitude of the two are independent. Because the mechanism of secondary compression is not well understood, secondary compression settlement calculations are understood to be only general approximations. If secondary compression is considered, the coefficient of secondary compression will increase with increasing consolidation pressures.

7.7 TIME DEPENDENT RATE OF CONSOLIDATION SETTLEMENT

As previously discussed, two distinct components in the analysis of consolidation settlement are magnitude and time rate. The following subsections address methods for computing estimates of the time rate, or required time, of long-term settlements due to the consolidation process. The time rate of consolidation settlement often controls the design and construction over the magnitude of consolidation settlement. Therefore, it is necessary to account for the consolidation rate in the design and/or construction schedule of a project and accommodate or address the time dependency as dictated by site requirements and construction needs.

For plastic fine-grained saturated soils, when a load is applied and overburden stresses increase, the low soil permeability resists the expulsion of water. And since water is incompressible, there is no instantaneous volume change (no reduction in void space). The result is an increase in pore pressure (u) beyond hydrostatic levels. This is termed excess pore water pressure (Δu). The instantaneous increase in pore pressure will equal the increase in overburden pressure from the added load ($\Delta u = \Delta \sigma$). As load continues to be applied, and overburden stresses continue to increase, excess soil pore pressures continue to increase. Under static conditions, soil effective stress (σ') is equal to total overburden stress (σ) minus hydrostatic stress (u). As excess pore pressure increases, the effective stress in the soil mass continues to decrease. With a decrease in effective stress comes a decrease in soil shear strength. The relationship between soil effective stress, excess pore pressure, overburden stress and hydrostatic stress is defined by the following equation:

$$\sigma' = \sigma - u - \Delta u$$

where,

σ' = soil effective stress
σ = total overburden stress
u = hydrostatic stress
Δu = excess pore pressure

If the rate of the load increase is not controlled, or the soil mass is not allowed adequate time to drain during loading, then a continued increase in excess pore pressure can result in catastrophic loss in soil shear strength. Bearing capacity and/or slope failures can, and have, resulted from not allowing excess pore pressures to dissipate sufficiently. The rate that load can be safely applied is a direct function of soil permeability and the maximum length of the drainage path pore water must take to be expelled from the soil mass.

Allowing excess pore pressures to adequately dissipate has two impacts to the soil mass. The first is the volume reduction of soil voids resulting in settlement. The second involves the increase in effective stress. As excess pore pressures dissipate, the effective stress not only increases but surpasses preload levels due to increase overburden stress. The effective stress increase directly impacts soil shear strength. As effective stress increases, shear strength increases by mobilizing more of the frictional component of soil shear strength. This is most easily observed in the soil shear strength equation: $S = c + \sigma' \tan ø$. The reduction in soil voids pushes soil particles tighter together having the dual impact of volume reduction (settlement), and higher interparticle stresses (increased effective stress), increasing shear strength.

To properly address design and construction needs, it is first necessary to develop a reliable estimate of the rate of consolidation settlement. The two general approaches available to develop a reliable consolidation rate estimate are laboratory testing of undisturbed samples and in-situ field testing.

7.7.1 Time Rate of Consolidation – Lab Testing

The most common approach of computing estimates of the rate of consolidation is laboratory testing of undisturbed samples using the one-dimensional consolidation test. As shown in Section 7.5.1, reduction of one-dimensional consolidation test results allows determination of coefficients used to estimate the magnitude of various phases of consolidation settlement. Similarly, consolidation test data can be reduced and presented graphically to produce a coefficient to estimate the time rate of consolidation. The value is termed the Coefficient of Consolidation (c_v). The procedure for determining the value of c_v and calculating the time for consolidation, using one-dimensional consolidation test data, is presented as follows.

1. Immediately after application of load, excess pore water pressure develops, and the soil has undergone no (0%) consolidation. The pore water pressure will then immediately start to dissipate at the drainage boundaries. The drainage boundaries are located at the interface with adjacent pervious layers. The further away from the interface with pervious layers, at interior points, the pore water pressure dissipates more slowly with time depending on the permeability of the compressible soil.

2. At any time after application of a load, the degree or percentage of consolidation ($U\%$) at a given depth is defined in the following equation:

$$U\% = \left(\frac{\Delta u_i - \Delta u}{\Delta u_i}\right) 100$$

where,

$U\%$ = degree or percentage of consolidation
Δu = the excess pore water pressure at that depth at that time
Δu_i = the initial excess pore water pressure immediately after load is applied.

The initial excess pore water pressure equals the total stress increment, Δp_t. When $\Delta u_i = \Delta u$ (i.e., at the instant of loading), the percent consolidation is zero. When $\Delta u = 0$ (i.e., at the end of consolidation), the percent consolidation is 100. This relationship is valid at any depth within the consolidating layer at any time from the instant of loading to the completion of primary consolidation.

The magnitude of consolidation settlement at any time after the application of load is directly proportional to the percent of consolidation that has taken place up to that time. The percent of consolidation ($U\%$) at any time (t) can be defined as the ratio of the settlement at that time (S_t) to the settlement at the end of primary consolidation (S_{ult}), or:

$$U\% = \left(\frac{S_t}{S_{ult}}\right) x\ 100$$

where,

$U\%$ = percent consolidation
S_t = settlement at time t
S_{ult} = the ultimate settlement

When S_{ult} is reached, excess pore water pressures are zero throughout the consolidating layer. This relationship is used to develop a "settlement-time curve" as shown in Figure 7.7.1-1. This curve shows the average degree of consolidation ($U\%$) corresponding to a normalized time expressed in terms of a time factor (T_v), calculated as follows:

$$T_v = \frac{c_v t}{H_d^2}$$

where,

T_v = time factor
c_v = coefficient of consolidation (ft^2/day)
t = time (day)
H_d = longest distance to a drainage boundary (ft)

Any consistent set of units can be used in the above equation since T_v is dimensionless. The longest drainage distance (H_d) of a soil layer confined by more permeable layers on both ends (two-way drainage) is equal to one-half of the layer thickness ($H_d = H/2$). When confined by a more permeable layer on one side and an impermeable boundary on the other side (one-way drainage), the longest drainage distance is equal to the layer thickness ($H_d = H$). The value of the dimensionless time factor T_v is indicated in Table 7.7.1-1 for various percentages of consolidation ($U\%$).

Table 7.7.1-1 – Average Degree of Consolidation (U%) vs. Time Factor (Tv)

U%	**T_v**	**U%**	**T_v**
10.0	0.008	80.0	0.567
20.0	0.031	90.0	0.848
30.0	0.071	93.1	1.000
40.0	0.126	95.0	1.163
50.0	0.197	98.0	1.500
60.0	0.287	99.4	2.000
70.0	0.403	100.0	Infinity

Notes: 1. Modified from FHWA, 2006

By rearranging the time factor (T_v) equation, the time (t) it takes for any given percent of consolidation (percent of total settlement) to occur, can be calculated as shown below.

$$t = \frac{T_v H_d^2}{c_v}$$

where,

t = time (day)
T_v = time factor
H_d = longest distance to a drainage boundary (ft)
Note: for sample height H, $H_d = H$ for one way drainage, and $H_d = H/2$ for two way drainage.
c_v = coefficient of consolidation (ft^2/day)

As observed from the equation, the time for any given percent consolidation, is a function of the longest distance or path to a drainage boundary. By using the consolidation time equation with the time factors for the indicated values of percent consolidation in Table 7.7.1-1, a predicted settlement-time curve can be developed as shown in Figure 7.7.1-1.

Figure 7.7.1-1 – Typical Settlement Time Curve

Notes: 1. Adapted from FHWA, 2006

The consolidation laboratory test data is used to determine the coefficient of consolidation (c_v) in the time for consolidation (t) equation. A graphical procedure using the log of time is commonly used. The method is shown in Figure 7.7.1-2 accompanied by the step-by-step procedure. The method uses the oedometer dial reading versus log of time plot of the laboratory consolidation test result data. The procedure follows the FHWA, 2006 reference. However, before presenting the step-by-step procedure, the following paragraph provides a brief discussion on selecting the appropriate load increment(s) used in the procedure.

AASHTO T216 is the “Standard Method of Test for One-Dimensional Consolidation Properties of Soils” referenced in Chapter 4 of this publication. The test method indicates the standard loading schedule must consist of a load increment ratio (LIR) of one that is obtained by doubling the pressure on the soil for each load increment. For reference, at a unit weight of 120 pcf, 4,000 psf is equivalent to approximately 33 feet of fill material. When anticipated design loads are anticipated to exceed 4,000 psf, test load increments should increase until they exceed anticipated design/construction loads. For additional information concerning the load schedule and consolidation testing, refer to Chapter 4. Whenever possible, lab testing must always be conducted in a manner modelling actual site and proposed construction and service conditions as closely as practical (see Chapter 4, Section 4.8.4). For consolidation testing, the doubling load schedule should be conducted to a stress/load level bracketing anticipated and/or design field conditions. To generate a consolidation curve with a straight-line portion (virgin compression) bracketing design loads, the load schedule should extend to a stress condition two load points beyond anticipated maximum stress conditions within the layer being tested. The load increment used for determination of the coefficient of consolidation should be consistent with maximum anticipated stresses in the layer being investigated and should be within the straight-line portion of the "e-log p" consolidation curve.

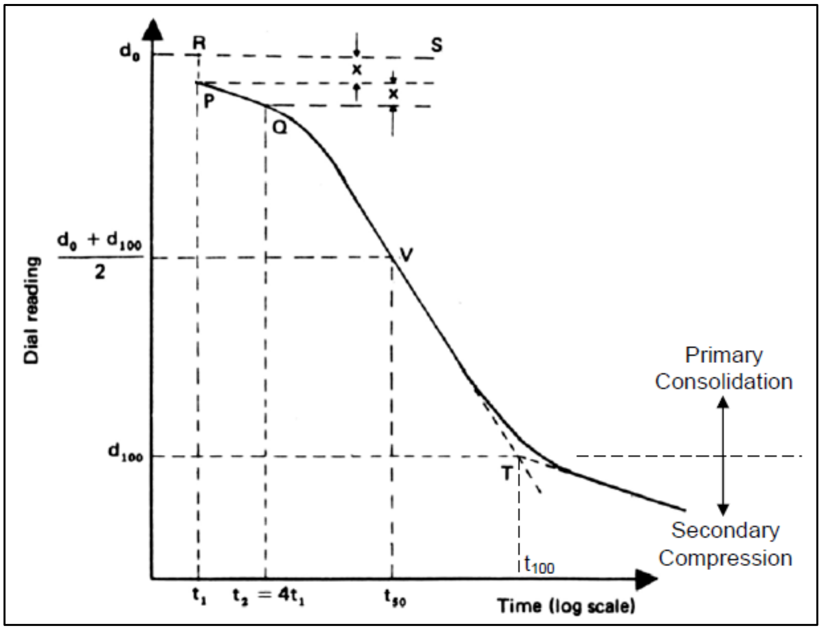


Figure 7.7.1-2 – Graphical Procedure for Determining Coefficient of Consolidation

Notes: 1. Adapted from FHWA, 2006

The procedure for determination of the Coefficient of Consolidation (c_v) is as follows and as shown in the above figure:

1. Select the appropriate load increment to determine the Coefficient of Consolidation (c_v).
2. Plot the dial readings of sample deformation vs. time for the selected load increment against time on a semi-log scale. "Dial readings" on y-axis and "time (log scale)" on x-axis.
3. Plot two points marked *P* and *Q*, on the upper portion of the consolidation curve that corresponds to times t_1 and t_2 respectively. Select points *P* and *Q* such that $t_2 = 4t_1$.
4. The difference in the dial readings between points *P* and *Q* is equal to *x*. Locate point *R* at a distance equal to the value of *x* above point *P*.
5. Draw a horizontal line through point *R* (line R-S). Project line R-S through the y axis (dial reading). The value of dial reading where the horizontal line R-S intersects the y-axis is identified as d_0, the dial reading estimated as corresponding to 0% consolidation.
6. Project two straight-line segments from both the primary portion of the consolidation curve and the straight-line segment of the flatter secondary

compression portion of the curve (located at the end of the curve). Mark the location where the straight-line segments intersect as point *T*.

7. Project a horizontal line back from point *T* to the y-axis. The value for the dial reading where the horizontal projection back from point *T* intersects the y-axis is identified as d_{100}, the dial reading estimated as corresponding to 100% consolidation. Projecting vertically downward from point *T* to the x-axis provides the estimated value of t_{100}, the estimated time for 100% consolidation.
8. Any sample deformation beyond t_{100} is considered as secondary compression (refer to Section 7.6).
9. Calculate the value of d_{50} using the following equation:

$$d_{50} = \frac{d_0 + d_{100}}{2}$$

10. Locate the calculated dial reading value of d_{50} on the y-axis. Project a horizontal line for the calculated value of d_{50} on the y-axis over to the consolidation curve. Mark the location where the horizontal line intersects the consolidation curve as point *V*.
11. Project a vertical line down from point *V* to the x-axis. The value of time where the vertical projection from point *V* intersects the x-axis is the value for t_{50}, the estimated time for 50% consolidation (0.197 is the time factor for t_{50}).
12. Calculate the value for the Coefficient of Consolidation for the soil layer using the equation below:

$$c_v = \frac{0.197 H_d^2}{t_{50}}$$

where,

c_v	=	coefficient of consolidation (ft^2/day)
0.197	=	time factor for 50% consolidation (T_{50})
H_d	=	longest distance to a drainage boundary (ft); Note: for two-way, drainage is equal to half the thickness of the layer being analyzed (*H/2*)
t_{50}	=	time for 50% consolidation (day)

With a value for c_v determined from the laboratory test data, the time for settlement can be determined for any percent consolidation using the time for consolidation equation and the appropriate time factor (T_v) from Table 7.7.1-1 presented above.

7.7.2 Time Rate of Consolidation – Field Testing

One field test to assess the time rate of consolidation is the piezocone penetrometer test (CPTU). The electric cone penetrometer is a device that is pushed into the ground to provide site characterization and empirical correlations for a variety of soil parameters. If the device also incorporates a piezometer to continually monitor pore pressures, it can be used to estimate the time rate of consolidation. Figure 7.7.2-1(a) shows an overview of the CPTU. It is useful for a range of subsurface materials including clays, silts, sands, and organic deposits. Figure 7.7.2-1(b) shows an assortment of electric cone penetrometer testing devices. They consist of a conical tip and a friction sleeve. With the use of an array of electric strain gauges, the devices measure both tip and side resistance during penetration, while the piezometer measure pore pressure.

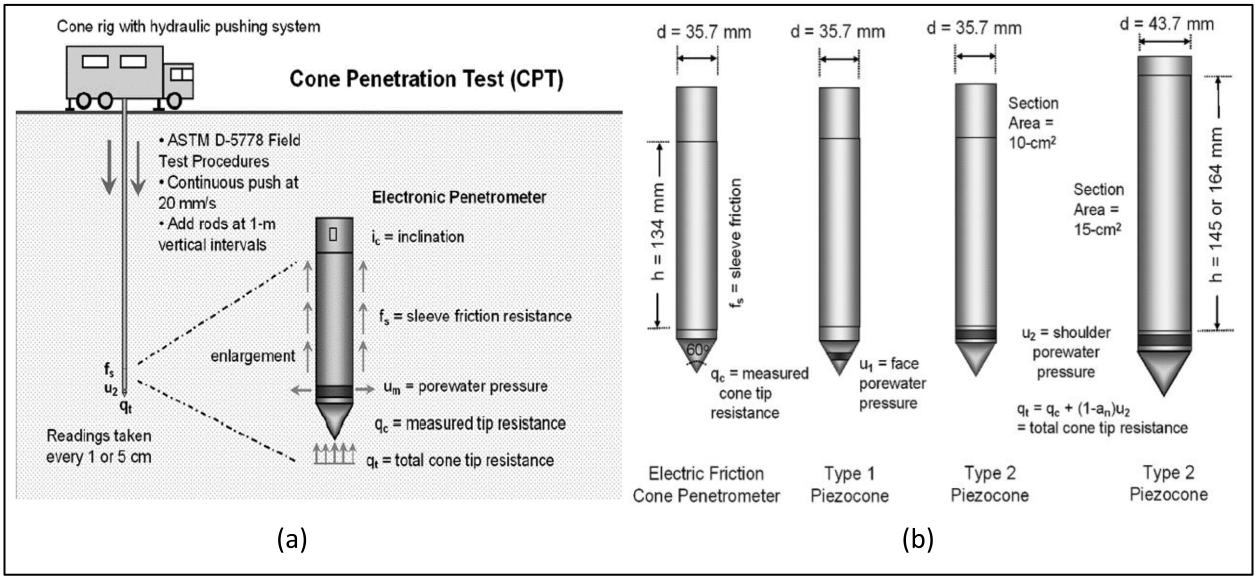


Figure 7.7.2-1(a) and (b) – Overview of the Cone Penetration Test and Testing Devices

Notes: 1. Modified from NCHRP Synthesis 368, 2007

A porous element is incorporated into the device on or behind the cone penetrometer tip. The porous element allows soil pore pressures to be transmitted to the piezometer mounted inside the body of the cone penetrometer. As the penetrometer is advanced into a soil, there is local disturbance of the soil as material is displaced around the penetrometer. If the material is at or near a condition of saturation, excess pore pressures are generated during penetration and displacement of the soil.

The excess pore pressures generated during penetration are due to the load imparted on the soil as it is displaced during penetration. The rate of dissipation of the excess pore pressures is a function of soil permeability and is time dependent. Therefore, the rate of pore pressure dissipation is directly related to the time rate of consolidation and the coefficient of consolidation. In order to determine this relationship, pore pressure dissipation (PPD) tests are conducted during CPTU testing. Pore pressures are monitored during penetration. When excess pore pressures develop during penetration, consideration must be given to running a PPD test. Determination for running a PPD test is based upon several factors:

1. Have the subsurface conditions and strata already been adequately defined and delineated from previous subsurface explorations?
2. If so, have the layers of concern and their depths and thickness been identified?
3. Has any laboratory testing been conducted on samples collected from previous explorations or subsurface borings?

Ideally, completed subsurface borings and lab testing will identify locations and depths of materials with the potential for consolidation settlement. These borings will also allow estimation of the groundwater depth. Knowledge of the depth of the phreatic surface is important to establish a threshold value for excess pore water pressure. When approximate strata locations and depths are anticipated, pore pressures and depth of penetration are closely monitored. As penetration approaches these zones, and high excess pore pressure readings are observed, PPD testing can be initiated.

In some situations, CPTU is conducted for a broad site characterization. In these instances, there may be either no knowledge, or only general indication, of the presence of strata subject to consolidation settlement. When this occurs, it is prudent to continuously monitor pore pressures and determine if the presence of layers subject to time dependent settlement could potentially present a problem. If pore pressures start to elevate beyond a value that is possible for the current depth of penetration (i.e., beyond hydrostatic pressure levels or more than 0.43 psi per foot of penetration depth), it is a good indication that the cone tip is in a saturated, low permeable layer and a PPD test should be initiated.

The start of a PPD test can begin at any time or depth when excess pore pressures are observed or suspected. The test is initiated by putting the CPT equipment in stand-by mode and switching to continuous monitoring and collection of pore pressure data. Data points of pore pressure are usually collected automatically at regular intervals (e.g., every five seconds, etc.). Again, information as to the depth of the phreatic surface is important to establish when pore pressures are in excess of phreatic pressures.

If the depth of the phreatic surface is not known, determination can be made from the pore pressure value of a PPD test that is run to 100% dissipation (i.e., the pore pressure dissipation curve becomes flat, indicating no change with time). Based upon the remaining pore pressure, the depth to the phreatic surface can be calculated. Water pressure increases with depth at a rate of 0.43 psi per foot of depth. Dividing the remaining value of pore pressure by 0.43 provides the height of the phreatic surface above the current depth or elevation. Subtracting the height of the phreatic surface above the current depth from the now current depth, provides the depth of the phreatic surface. This value can be used for analysis of PPD tests at other depths in the same cone penetration test, or local CPT's with PPD tests conducted in conditions with little or no groundwater gradient.

As can be observed in Figure 7.7.2-1(b), the two possible locations for the piezometer porous element include the face of the conical tip (u_1 position) and immediately behind the conical tip (u_2). Some models of testing equipment allow the use of either option. It is recommended to conduct testing with the porous element on the face of the tip, or the u_1 position, whenever possible. This is recommended because negative pore pressures can be generated in some soil types with the porous element in the u_2 position. The plotted curve shape for normalized pore pressure can also be different, depending upon soil type location of the porous element. The normalized pressure can sometimes first show an increase before dissipation initiates. While the data is still valid, it complicates the analysis. Refer to "An approach to evaluation of Field CPTU dissipation data in overconsolidated fine-grained soils" for the description and plotting of PPD test data for different soil stress histories and porous element locations.

There are various methods of presentation and plotting of PPD test data. These include pore pressure (PP) vs time, PP vs log time, and PP vs square root of time. Figure 7.7.2-2 shows a typical example of each type of plot. Depending upon the character of the material, one method may provide a more useful presentation than another. Refer to the Sully document indicated in the previous paragraph for information concerning various plotting methods. PP vs square root of time provides a curve more clearly depicting pore pressure dissipation (see Figure 7.7.2-2(c). Regardless of the method of presentation, the initial presentation consists of the raw pore pressure test data.

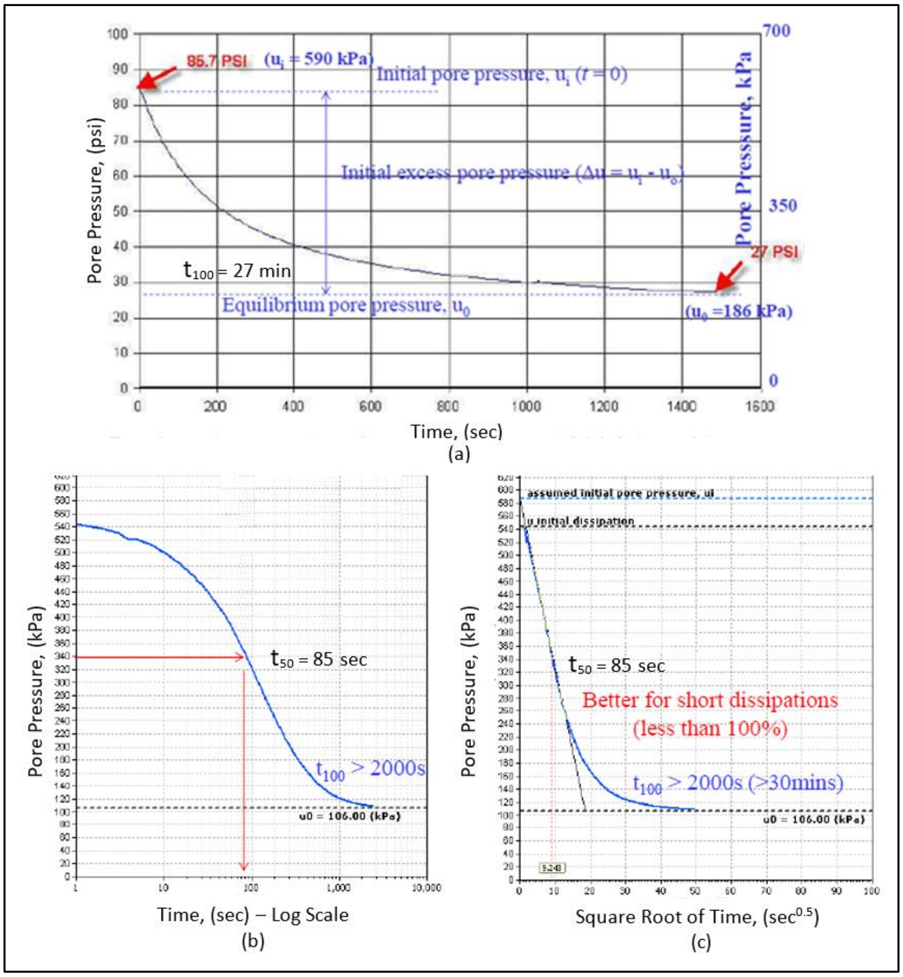


Figure 7.7.2-2 – Typical Pore Pressure Dissipation Curves

Notes: 1. Modified from Robertson, 2013

Based upon the depth of the phreatic surface, the PPD test data can then be normalized as shown in Figure 7.7.2-3 and constructed from Figure 7.7.2-2(a). A normalized presentation of the data simplifies analysis and calculation of the coefficient of consolidation, although this raw dissipation curve can also be used. The normalized plot of PPD data is used to determine the t_{50} value, the time to 50% consolidation, for the Coefficient of Consolidation for horizontal drainage (c_h) calculation. Note that the curve can be used to estimate the time to any percent consolidation.

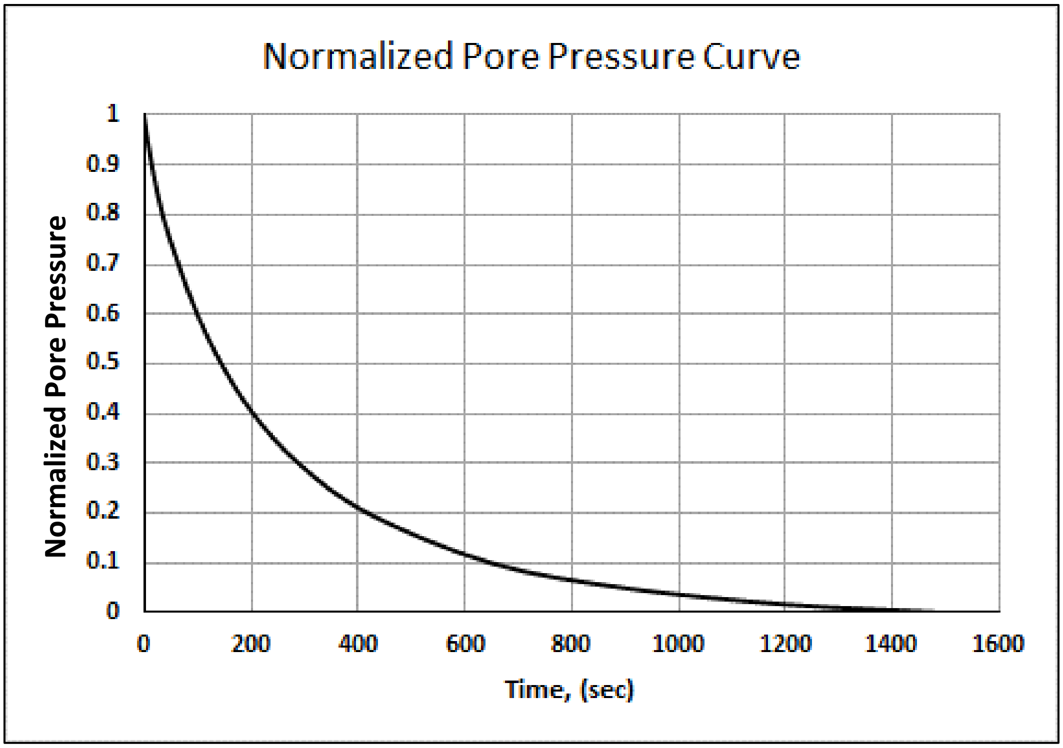


Figure 7.7.2-3 – Normalized Pore Pressure Dissipation Curve

There are various equations proposed to calculate coefficient of consolidation using PPD test data from the cone penetrometer. Some equations use values for soil properties that may not be readily available and/or must be estimated. In such cases, a simplified form of the equation can be used. This equation is similar to the equation used for one dimensional laboratory consolidation testing. The equivalent equation for coefficient of consolidation using PPD test data from CPTU testing is as follows:

$$c_h = \frac{T_{50}^{*} r^2}{t_{50}}$$

where,

c_h = Coefficient of consolidation for horizontal drainage
T_{50}^{*} = Modified time factor (for 50% consolidation) – see discussion for determination of T* below
t_{50} = Time for 50% consolidation
r = Radius of the cone penetrometer

Another common form of the this equation includes the soil rigidity index (I_R). Knowledge of the soil shear modulus (G) and the soil shear strength (S_u) is required to determine the I_R of the soil. Refer to NCHRP Synthesis 368, for additional information in determining the value of the rigidity index. The equation is very similar to the simplified form as follows:

$$c_h = \frac{T^*_{50} r^2 \sqrt{I_R}}{t_{50}}$$

where,

c_h	=	Coefficient of consolidation for horizontal drainage
T^*_{50}	=	Modified time factor (for 50% consolidation) – see discussion for determination of T* below
t_{50}	=	Time for 50% consolidation
r	=	radius of the cone penetrometer
I_R	=	Rigidity Index = G/S_u = Shear Modulus/Soil Shear Strength

The modified time factor, T^*, depends upon the location of the cone pore pressure element (i.e., midface position u_1 or shoulder position u_2). Figure 7.7.2-4 shows the relationship between normalized excess pore pressure and the modified time factor for the cone pore pressure element in the u_1 and u_2 positions. From the two figures the values of the modified time factor at 50% consolidation, are 0.121 and 0.250 for the u_1 and u_2 pore pressure element positions, respectively. Table 7.7.2-1 indicates the values of T^* at various normalized excess pore pressures for the u_1 and u_2 pore pressure element positions.

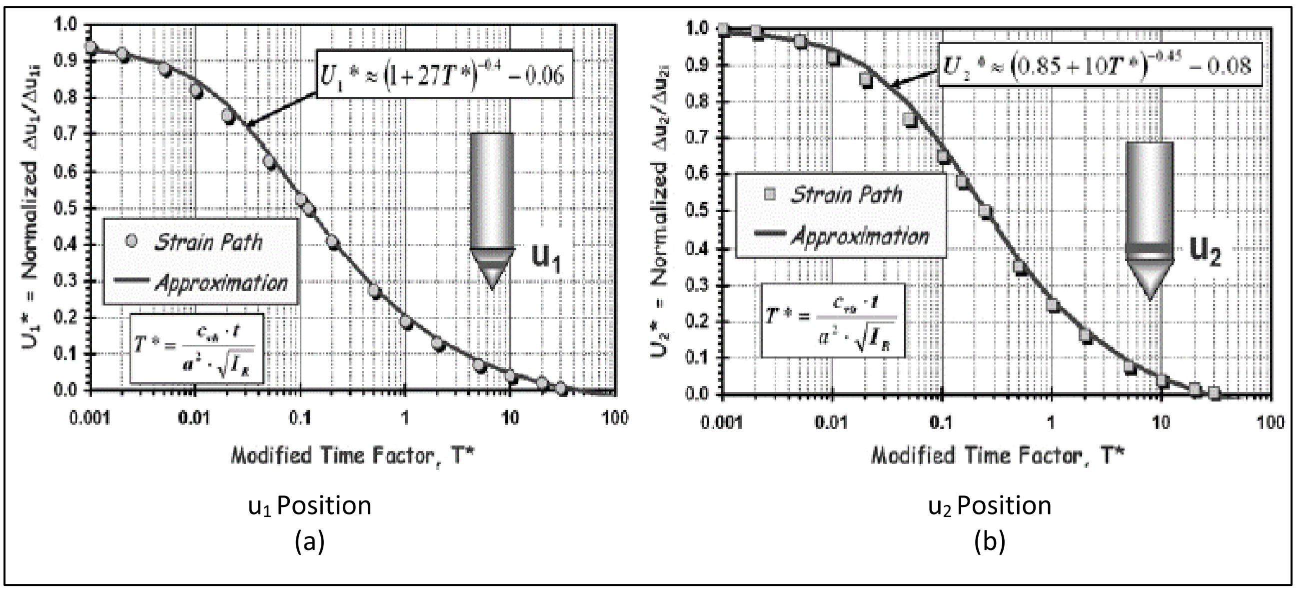


Figure 7.7.2-4 – Normalized Pore Pressure vs Modified Time Factor

Notes: 1. Modified from NCHRP Synthesis 368, 2007

Table 7.7.2-1 – Modified Time Factor (T*) Values for Cone Penetrometer PPD Test

Percent Consolidation	Normalized Pore Pressure	T* u_1 Position	T* u_2 position
10	0.9	0.004	0.020
20	0.8	0.017	0.048
30	0.7	0.037	0.089
40	0.6	0.068	0.151
50	0.5	0.121	0.250
60	0.4	0.221	0.426
70	0.3	0.439	0.773
80	0.2	1.04	1.61
90	0.1	3.58	4.43
95	0.05	9.19	9.22
99	0.01	41.96	27.29

As indicated for the second equation, the rigidity index (I_R) is equal to the shear modulus divided by the soil shear strength (G/S_u). Estimates of elastic modulus, Poisson's ratio, and the soil shear strength can be made through material or testing correlations to estimate the rigidity index. The shear modulus is related to the elastic modulus as follows:

$$E = 2G(1 + \mu)$$

therefore,

$$G = \frac{E}{2(1 + \mu)}$$

where,

E = Elastic Modulus
G = Shear Modulus
μ = Poisson's Ratio

The equations for coefficient of consolidation using PPD test data from CPTU testing are very similar to the equation used for one dimensional laboratory consolidation testing. One of the primary differences in the two methods is that for the lab test, drainage is in the vertical direction, while drainage during PPD testing with the cone penetrometer occurs radially in the horizontal direction. Since some soils, especially clays, exhibit differences in permeability in the vertical and horizontal direction, this anisotropy must be accounted for in determination of the coefficient of consolidation for drainage in the horizontal versus vertical directions.

A literature review was conducted to determine the types and conditions and levels of soil anisotropy may exist relative to the rates of horizontal and vertical consolidation. The consolidation anisotropy ratio is defined by the value k_r (with $k_r = c_h/c_v$), or the ratio of horizontal to vertical coefficient of consolidation. Values of k_r greater than 1.0 indicate that the horizontal coefficient of consolidation (c_h) is greater than the vertical coefficient of consolidation. Values of k_r typically range from 1.0 to 2.0; however, values of k_r as high as 40 have been observed for varved clays.

Varved clays are characterized by regular alternating layers of finer and coarser deposits. In general, the coarser and more homogeneous the deposit, the lower (k_r closer to 1.0) the consolidation anisotropy ratio, while the finer and more heterogeneous the deposit, the higher the consolidation ratio. There is also evidence that the value of k_r increases with increasing effective overburden stress. Except for varved clay deposits and in the absence of test data, to calculate the ratio of horizontal to vertical coefficient of consolidation, a value of $k_r = 1.6$ is recommended. While this may prove slightly conservative in some situations, the implication would simply be shorter consolidation times than anticipated. When varved or layered soil conditions are observed or indicated, consider conducting permeability testing in both the vertical and horizontal directions.

Once a value of k_r is determined, the vertical coefficient of consolidation, cv, can be calculated as follows:

$$c_v = \frac{c_h}{k_r}$$

where,

c_v = Coefficient of consolidation for vertical drainage
c_h = Coefficient of consolidation for horizontal drainage
k_r = Consolidation anisotropy ratio

With the value of the coefficient of consolidation for vertical drainage determined, the calculations for determining the time rate of consolidation can proceed as indicated in Section 7.7.1 for lab testing.

7.8 PROCEDURE TO ESTIMATE CONSOLIDATION

The step-by-step process for estimating the magnitude of and time for consolidation to occur for a single-stage construction of an embankment on soft ground is outlined as follows:

1. From laboratory consolidation test data, determine the “e-log p” curve and estimate the change in void ratio that results from the added weight of the embankment. Create the virgin field consolidation curve by using the guidelines presented in Table 7.5.1-1.
2. Determine if the foundation soil is normally consolidated, overconsolidated or underconsolidated.
3. Use equations presented in Section 7.5.2 to compute the primary consolidation settlement for normally consolidated, overconsolidated or underconsolidated foundation soils.
4. Determine c_v from laboratory consolidation test data. Two graphical procedures commonly used for this determination are the logarithm-of-time method (log t) and the square-root-of-time method. The log of time method is presented in Figure 7.7.1-2. Because the two methods are different approximations of theory, they do not provide identical answers. Often the square root of time method gives slightly greater values of c_v than the log t method. Alternately, use CPTU and PPD data to estimate the value of c_v.
5. Use the equation presented in Section 7.7.1 to calculate the time to achieve 90% - 95% primary consolidation.

7.9 CALCULATING SETTLEMENT

Determining when and where to run calculations to estimate settlement for proposed construction is a function of the subsurface conditions and the proposed construction. The level of thoroughness and consideration dedicated to the process of estimating settlement is also a function of these same two factors. Any situation involving or requiring greater attention to potential impacts from settlement, dictates a proportionally more thorough settlement analysis. This may include a more thorough subsurface exploration (e.g., number of borings, etc.), increased level of laboratory testing, in-situ field testing, a more careful and rigorous modeling and analysis, or, when applicable, a design of construction monitoring and/or control measures.

7.9.1 When to Calculate Settlement

In general, the greater relative density or stiffness of the material encountered in the subsurface, the lower potential for settlement under applied loads. Competent rock is typically not a concern with regards to settlement because of its relatively high strength and modulus. However, applicable sections of DM-4 must be followed for any requirements concerning settlement of foundations on rock. This includes, but is not limited to, Sections 10.4.7.2.4P, 10.4.7.3.1P, and 10.6.2.4.4. Soil materials must be scrutinized with greater care. For granular or cohesionless materials, the lower the SPT N-value, the lower the relative density and higher potential for settlement. For clayey or cohesive soils, the lower the SPT N-value, the less stiff or softer the material and the higher the potential for settlement.

The primary difference with regards to settlement between cohesionless and cohesive soils is whether anticipated settlements will be elastic or time dependent. Elastic settlement occurs rapidly with applied load and is associated with cohesionless soils or low percent saturation cohesive soils, while consolidation settlement is associated with nearly saturated or fully saturated cohesive soils that require time for excess pore pressure to dissipate after loading is applied. As the excess pore pressure dissipates, load is transferred from the pore water to the soil skeletal structure. The Deformation, observed as settlement, occurs as the load is transferred to the soil structure. The different modes of soil deformation and settlement are covered in greater detail in previous sections of the chapter.

It is also important to recognize that materials controlled by time dependent consolidation settlement, frequently will have an elastic settlement component. This component must be considered in any analysis conducted to estimate the total magnitude of anticipated settlement. Whatever the mechanism of soil deformation and settlement, a careful analysis of subsurface conditions allows an assessment of the potential for, and anticipated magnitude of, foundation settlement with applied loads.

Separate from the potential for a material to experience deformation (settlement) under load, is whether the mechanism and magnitude of settlement presents a problem. This is a function of the proposed construction. Simply put, rigid structures are more sensitive and prone to damage than flexible structures. For highway environments, rigid structures include bridges, retaining walls, culverts, pipes, sound barriers and sign structures. Flexible structures include embankments and reinforced slopes. Some structures, such as mechanically stabilized earth walls, include both flexible and rigid components. Therefore, such intermediately rigid structures may have an intermediate level of sensitivity to settlement and is further discussed in Section 7.11.1. Follow DM-4 requirements for differential settlements for MSE walls and MSE wall facings.

Complete a more detailed settlement analysis for structures that have a high degree of sensitivity to foundation settlement. Typically, bridge structures and retaining walls having shallow rigid foundations could be highly sensitive to settlement and more significantly, differential settlement between support units as further discussed in Section 7.11.1. Subsurface conditions resulting in potential for high differential settlement would also warrant closer attention and more thorough exploration and analysis. Follow DM-4 requirements for allowable settlement for shallow foundations, including requirements for addressing angular distortion between supporting units.

Situations of foundation settlement where less scrutiny might be exercised may include highly flexible structures, which by nature do not have the same level of sensitivity or are generally not subject to the same level of risk. This is not to imply that the potential for, and possible impacts of, foundation settlement can be ignored for more flexible and less settlement sensitive structures, but rather a less rigorous plan of exploration, testing, and analysis might be warranted. Immediate (elastic) settlement is also typically of less concern than consolidation settlement since elastic settlement occurs rapidly as load is applied.

Alternately, there are some instances where a more thorough exploration and testing program, and more careful settlement analysis, must be observed for even very flexible structures, such as embankments constructed overlying thick deposits of highly compressible soils that often involve consolidation settlement. The anticipated magnitude of settlement for such conditions may be so high, that even a low percent of anticipated deformation, may result in damage to secondary structures such as drainage or pavements.

Any situation where time dependent consolidation settlement may impact the flow of work, ultimately has the potential for costly delays and associated delay claims by contractors. In such situations, explicit specification language in combination with instrumentation, may be used to control the flow of work and avoid unanticipated construction delays or associated claims.

In general, rigid structures with shallow foundations on highly compressible soils, soils with variable compressibility, or compressible soils with variable thickness present a high risk for potential problems and must be addressed with great scrutiny. Compressible soils include low SPT N-value, soft, fine-grained materials such as clays and silts, or very loose, granular fine sands. Soft, saturated clays and silts with sufficient plasticity may present a problem with both time rate and total magnitude of settlement. Variable compressibility can result from compressible soil deposits of variable thickness, or deposits with N-values indicating variable consistency or density. For structures with deep foundations through compressible materials, downdrag may be of concern. Time rate of consolidation is of concern for embankments constructed of soft, saturated, clayey materials. Quality control measures may be necessary to stage construction to prevent foundation failure and consequential embankment slope failure.

7.9.2 Where to Calculate Settlement

Having discussed when to address settlement, it is also important to consider where, within the footprint of a structure, settlement calculations may need to be conducted. Typically, for any large footprint, rigid or flexible structure, settlements are calculated at the center of the structure and the corner. This is performed both to verify differential settlement is not an issue for rigid foundations, and to ensure total and differential settlements are not an issue for secondary structures (e.g., box culverts, pipes, pavements, etc.) Additionally, settlement may need to be calculated at multiple corners of rigid structures if highly variable conditions exist beneath the footprint of rigid foundations. Service loads would typically govern for settlement calculations; however, DM-4 and AASHTO LRFD bridge design specifications must be reviewed for other governing cases and any additional requirements concerning settlement analyses for structures.

For small footprint structures, settlements are typically calculated at the center of the structure unless specific subsurface, site or construction conditions dictate otherwise. Settlement may need to be calculated for any structure at specifically identified critical locations. Such critical locations may be the result of subsurface conditions, unusual site conditions or structure loadings, other sensitive or at-risk surface or buried structures, or are identified as a result of the proposed construction sequencing or loads.

As is evident by the above statement, thoroughly consider assessing the impacts of potential load induced settlement to all components and phases of a construction project. This is not intended to imply that every specific proposed construction element during every differential loading phase of construction requires a settlement analysis. Carefully consider the level of effort that must be taken to avoid costly construction and maintenance problems.

7.10 SETTLEMENT MONITORING

Settlement monitoring is a function of site-specific subsurface conditions/project needs and may include monitoring efforts both during and post-construction. Methods for monitoring settlement can range from relatively simple elevation monitoring of a strategically laid out array of points to more sophisticated approaches using strain gages or lasers. Each method has its own inherent advantages, disadvantages, weaknesses, and strengths. Settlement monitoring results must be presented, interpreted, and reported carefully and accurately. Direction or procedures are best provided in a monitoring plan. The following subsections provide an in-depth analysis of settlement monitoring and interpretation of the monitoring results.

7.10.1 Purpose of Settlement Monitoring

There are a variety of situations that may result in the need or desire to monitor settlement during and/or after construction. One common reason may be to verify design assumptions and estimated settlement to provide a level of assurance the final product will function as intended. Because the level of effort and cost to measure actual settlement of a foundation can be small (especially relative to the cost of the proposed construction), such measurements can provide a simple and effective means of verification to minimize risk and provide indication of any unforeseen problems or conditions. Such precautions may be especially prudent for critical and/or settlement sensitive structures, or when building next to structures that may be at risk of settlement induced by adjacent construction loads.

Another common reason to monitor settlement is for construction control when time dependent settlement is anticipated. As was previously discussed, due to the inherent variability of soil materials, calculated predictions for both settlement magnitude and time rate are estimated values. Consequently, ensure time dependent settlement is sufficiently complete before advancing with the next construction phase to prevent damage to subsequently constructed components.

A second, but equally valuable benefit to monitoring time dependent settlements, is to have adequate contractual control of construction activities. Including requirements in contract specifications for completion of time dependent settlement before initiation of subsequent phases of work provides the necessary information for contractors to bid work on an equitable basis, while protecting the owner from potentially costly delay claims. Requirements and expectations that are clearly indicated allow contractors to prepare bids and plan work accordingly, while assuring owners proper completion of work at a competitive cost.

7.10.2 Methods of Monitoring Settlement

Reliability, cost, and redundancy must all be considered in selection of the appropriate method of settlement monitoring. Reliability is an umbrella term covering the accuracy of the data and/or observations recorded, the repeatability of the system over multiple data sets or observations, and the long-term survivability of the system. Cost includes not only the initial investment in the required devices and/or equipment, but the cost of collecting and analyzing the information observed and recorded. Redundancy addresses the confidence with which decisions can be made with the observed and recorded information. This can be addressed with the use of multiple methods or technologies to monitor anticipated settlements, or the use of a redundant number of devices or data points in the array when using a single method or type of device. Selection of the most suitable method or methods is a balancing of the above factors against project needs, site conditions, the proposed construction, and most importantly, the risk associated with having unreliable information. In other words, what are the potential impacts and ramifications if no basis is present to make an informed decision or recommendation.

The benefits of an adequate settlement monitoring plan might best be described as a level of prudent effort and relatively small expenditure of the following:

1. Protecting a large dollar investment in highway infrastructure,
2. Providing insurance against potential construction cost overruns and claims
3. Preventing unnecessary highway user losses.

Generally, it is not necessary to employ a high level of sophistication and complexity to monitor settlement. Simplicity often provides more reliability at a lower cost, yielding sufficiently accurate and reliable data to make sound decisions while significantly reducing risk.

7.10.2.1 Settlement Monitoring Options

Of the variety of tools and techniques to monitor settlement, the most practical, efficient, and reliable is the settlement plate as shown in Figure 7.10.2.1-1. As shown in the figure, a settlement plate consists of threaded steel pipe welded to a rigid steel plate. The assembly is placed on a thin level lift of compacted fill immediately over top of the compressible layer that is to be monitored, before the placement of any fill. A PVC pipe sleeve is placed over the steel pipe to allow the pipe to move freely as the plate settles with the compressible layer beneath the plate. PVC and steel pipe sections are added as the height of the fill increases.

Once the plate is installed and before any fill is placed, the elevation of the top of the steel pipe is surveyed. This is done for each settlement plate installed for monitoring. As fill is placed, the top of the pipe elevation is surveyed on a regular schedule as directed in the settlement plate special provision. To determine the elevation of the top of the steel pipe, the installation must be compared to a stable benchmark of known or locally assigned elevation. The exact total length of the steel pipe and couplings must be known. As settlement occurs, the observed elevation at the top of the steel pipe is compared to the initial elevation at the top of the pipe. The difference between the current observed and initial elevations, is the amount of settlement of the foundation material beneath the plate. Because the top of the pipe must be surveyed to determine settlement, the lengths of steel and PVC pipe added must be known as the height of the fill increases. The top of the pipe must always remain readily and easily accessible for accurate observations and protrude above the PVC sleeve.

A minimum of two stable benchmarks must be established during the original installation of the settlement plate. This allows cross checking for potential errors and provides a level of redundancy should one of the two primary benchmarks be lost, disturbed or damaged. If a benchmark is lost, a replacement benchmark must be established immediately. It is good practice to require establishing a third benchmark during original installation, so the elevation of the reserve benchmark has already been established relative to the two primary benchmarks.

When surveying the top of pipe at each settlement plate, take readings relative to both of the primary benchmarks as a means of error checking. If a discrepancy arises, the reserve benchmark can be used to help resolve the problem. The benchmarks must be in protected areas, far enough away to be outside the area of influence of any settlement that occurs, but as close as reasonable to facilitate ease of surveying and adequate accuracy. The benchmarks should be stable hard points. However, if none are available at suitable locations, they can be constructed at convenient locations on the project site. Benchmarks should not be near each other, and preferably should be located on opposite sides of the area being monitored.

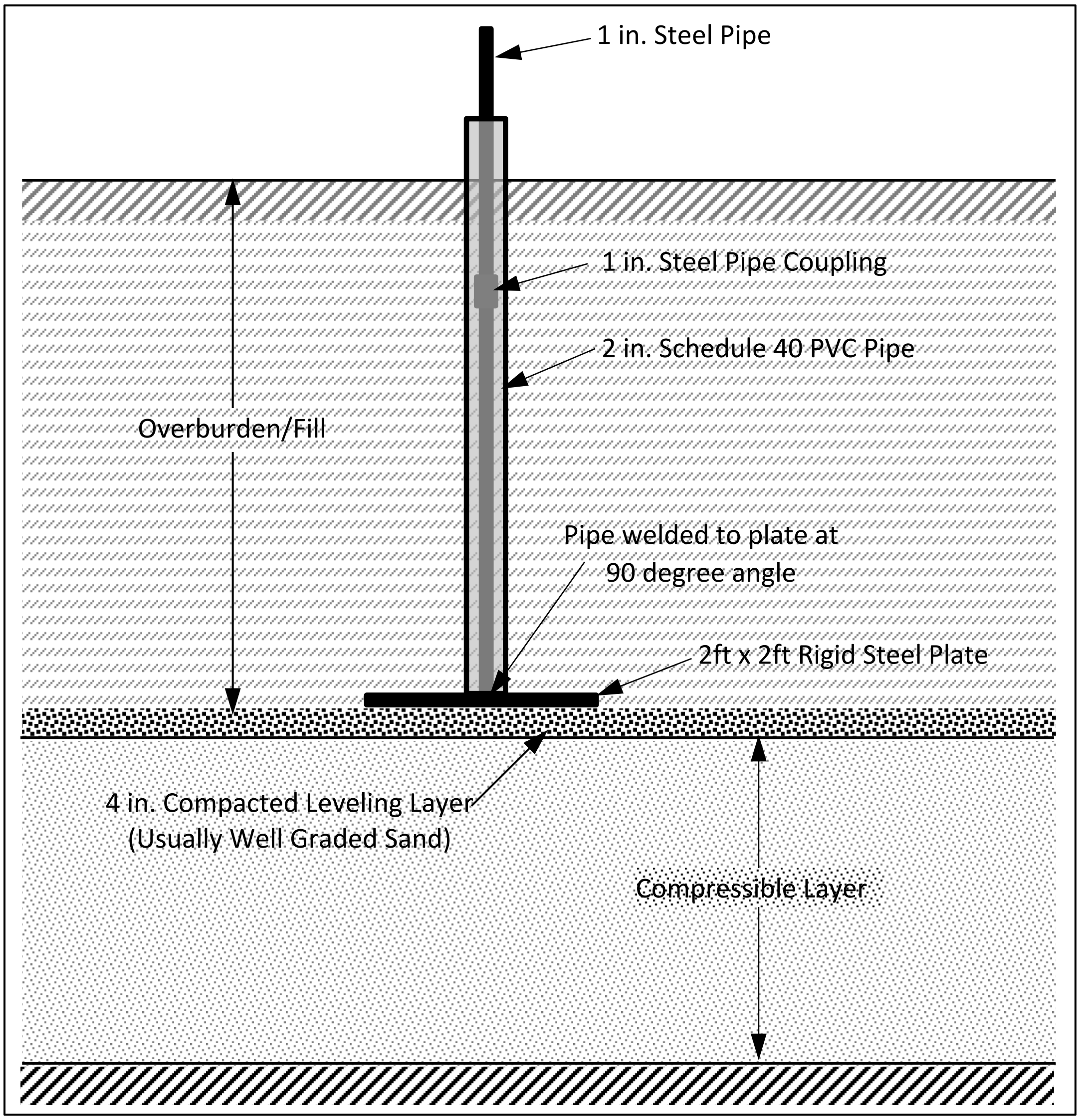


Figure 7.10.2.1-1 – Typical Settlement Plate

If the top of the compressible layer is not at the current ground surface elevation, there are several alternative options for monitoring settlement.

1. Install a settlement plate. If the top of the compressible layer is not at the current ground surface nor excessively deep, it may be acceptable to simply install the settlement plate at the current ground surface elevation. When considering this approach, attention must be given to the thickness and compressibility of the overlying layer, as to whether suitable results can be obtained.

2. Drill in a settlement point. The concept is identical to a settlement plate, except instead of a plate at the end of the assembly, an adequate length of steel pipe is grouted in place immediately above the compressible layer to be monitored. A series of very short lengths of pipe and couples, or other suitable mechanism, can be incorporated to adequately anchor the pipe. As with the settlement plate, a

PVC pipe sleeve is installed above the anchored portion of pipe to allow free movement. After grouting the initial length of steel pipe and installing the PVC pipe sleeve, the process is identical to that of a settlement plate.

3. Use prefabricated push or drilled in settlement points. These devices perform identically to settlement plates or grouted pipes. They have an inner and outer pipe and are either pushed or drilled to the desired elevation. Once at the desired elevation, the center rod is pushed extending a pronged anchor into the soil immediately below as shown in Figure 7.10.2.1-2 from Durham Geo. Several companies carry these types of settlement points including Geokon, Durham Geo, and Geonor. Once installed, these devices are maintained and monitored just like a settlement plate.

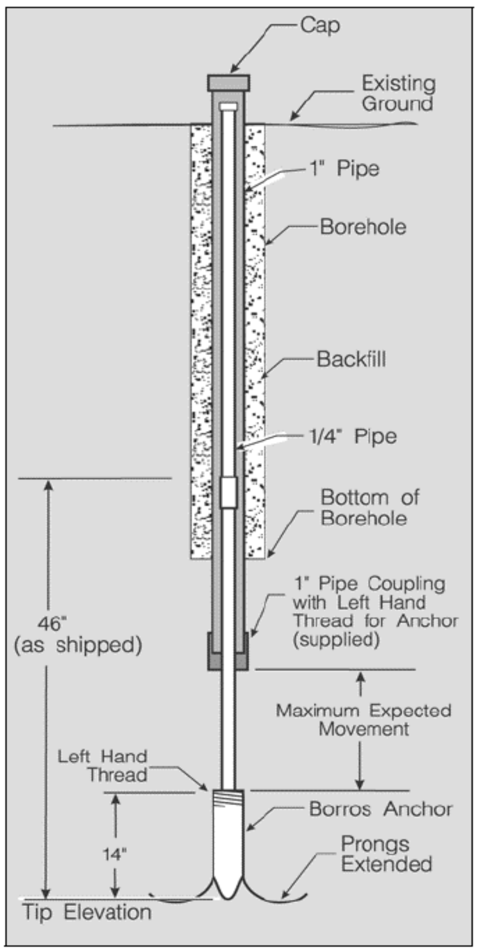


Figure 7.10.2.1-2 – Typical Settlement Point with Borros-Type Anchor

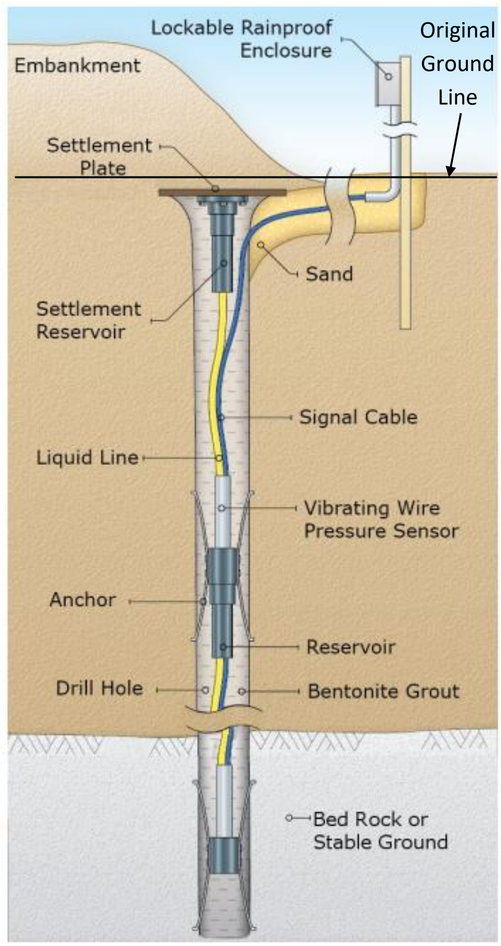


Figure 7.10.2.1-3 – Typical Vibrating Wire Settlement System

An alternative option to settlement plates is use of a vibrating wire piezometer as shown in Figure 7.10.2.1-3 The piezometer is connected to a fluid reservoir with a tube. The fluid reservoir is attached to a settlement plate. A signal cable is also attached to the piezometer. The piezometer is placed into stable ground that will not experience settlement under load. Typically, a hole is drilled to rock and the piezometer is embedded into rock using a weak bentonite grout.

The fluid reservoir is located above the piezometer near the ground surface, with sufficient cover to prevent any temperature changes to the fluid reservoir after installation. It is recommended that electronic temperature sensors be installed both with the reservoir and the piezometer to monitor temperature around the system components. In addition to monitoring settlement, vibrating wire piezometers are also used to monitor pore water pressure, which is further discussed in Section 7.10.2.2.

It is crucial to strictly follow manufacturer instructions during installation of these systems. The reservoir must always be kept upright after preparing, or bleeding, the fluid reservoir to prevent any air from entering the lines. The entire system must be allowed to come to thermal equilibrium before making any baseline readings. The temperature sensors allow checking for thermal equilibrium before baseline readings, assuring that heat from hydration of the grout has dissipated. The sensor at the reservoir also allows monitoring of temperature for the life of the system to ensure any fluid pressure changes due to temperature fluctuations do not influence piezometer readings.

Install and properly maintain both survey monitored and piezometer type settlement monitoring devices so that they perform accurately and reliably. Surveyed monitored plates have the advantage of simplicity but have potential risk of damage from construction equipment. Piezometer type settlement plates have greater potential installation or functional problems, but once installed have little risk of damage. The type(s) of settlement plate(s) selected can be influenced by site conditions, depth to the compressible layer(s), experience of personnel, available equipment or user preference. Again, redundancy in the total number and different types used, is an important factor towards assuring a successfully implemented monitoring plan.

Redundancy, layout, and survivability must be addressed in preparing an instrumentation plan specifying a variety of available settlement monitoring devices. Redundancy addresses the reliability and confidence with which decisions can be made with the observed and recorded information. For example, relying upon information collected from a single settlement plate must never be done as there is no cross check, confirmation or reliability of the observed and recorded information. An array of settlement monitoring devices and locations would be necessary to adequately represent and confirm observations to provide confidence in, and support of, any conclusions and recommendations.

Layout is the number and pattern that the settlement plates or other monitoring devices are distributed to provide a reliable representation of the amount of foundation settlement at the site. Generally, the more critical or sensitive the structure, the larger the area of concern, and the more variable and complex the subsurface conditions (e.g., multiple compressible layers, compressible layers of variable thickness, etc.), the greater the number of settlement monitoring devices and monitoring locations that will be required.

A good starting point for the minimum number of settlement monitoring locations and devices is three per substructure. For a very long structure, for example a long wall or very wide embankment, the number would increase; however, the increase in the number of settlement monitoring locations is not necessarily proportional to the increase in the footprint area of the structure. It is necessary to have a sufficient number of monitoring locations to adequately monitor the progress of settlement, it is also important to keep the number reasonable without sacrificing reliability of the data and resulting recommendations.

An excessive number of settlement monitoring devices not only creates an unnecessary increase in costs (e.g., installation, monitoring, data analysis, etc.), but they could also impact fill placement and compaction activities, reducing project efficiency. These and other intangible factors need to be considered in the design of any instrumentation plan. While three is indicated as the minimum number of settlement plates per substructure, there may be some situations where it is appropriate to deviate from this minimum. Situations involving uniform subsurface conditions and relatively small structures (e.g., short span bridges, walls with small footprints, short culverts, etc.), may simply not have adequate space to incorporate three settlement monitoring locations to allow efficient and/or effective construction operations.

Finally, it is necessary to address and consider survivability. If properly fabricated and installed, settlement monitoring devices will function adequately for the required period of monitoring. Problems with improper functioning monitoring devices can be the result of construction activity. During the process of embankment construction, the traffic congestion of equipment (e.g., dump trucks, dozers, compactors, etc.) involved in moving, placing, spreading and compacting multiple levels of fill, can result in impact and damage to monitoring equipment. Therefore, employ a variety of measures to account for the risks to monitoring device survivability, as follows:

- Specify for the installation of enough settlement monitoring devices, so that the potential loss of any one device, still provides adequate number and spread of monitoring points to provide reliable results.
- Ensure that monitoring devices extend above the construction grade when necessary and are visible to equipment operators. Require that devices and locations be marked or painted bright, highly visible colors in a repeating contrasting pattern, and install flagging at the top of the device.
- Prepare the layout plan so that monitoring devices are aligned with one another and arranged as uniformly as possible, leaving adequate space for placement and compaction equipment along the outer edges of fill placement or in vicinity of other structures or obstructions as shown in Figure 7.10.2.1-4. Uniform alignment assists equipment operators in being able to run operations in a pattern without having to work around randomly scattered obstructions.

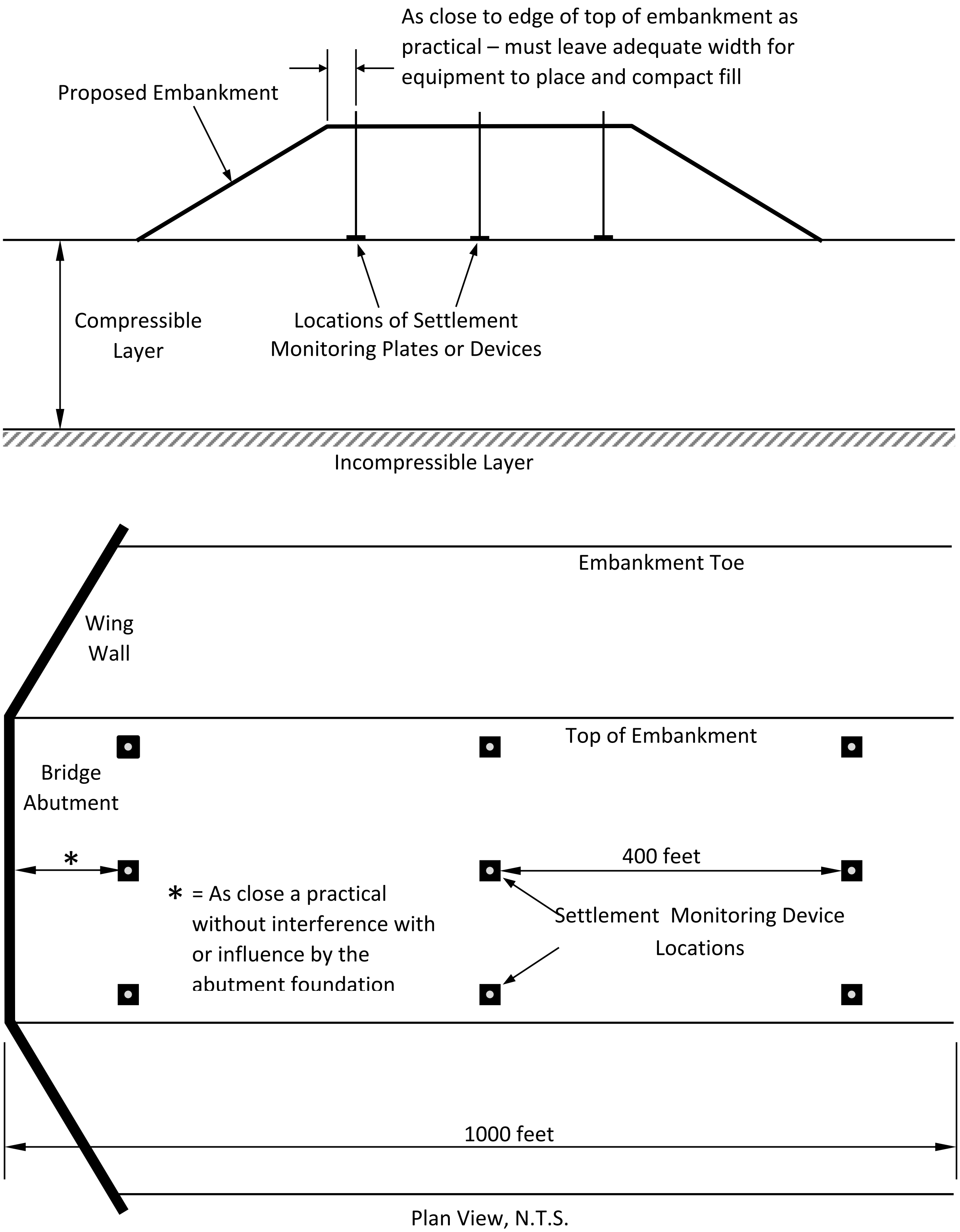


Figure 7.10.2.1-4 – Example Settlement Monitoring Layout for Embankment and Structure

7.10.2.2 Piezometers

When dealing with consolidation settlement, it is often the time required for settlement to occur that controls, as opposed to the total settlement magnitude. This is a function of excess pore pressure. Piezometers monitor soil pore pressure. While settlement plates are used for observing the total magnitude of settlement (i.e., elastic or consolidation), piezometers are used to monitor and assess the time rate of consolidation settlement. If embankment loads are applied too fast, excess pore pressure will continue to increase until there is enough loss of shear strength that failure of the foundation soil occurs. Foundation failure then progresses up into the embankment, usually resulting in slope failure. Therefore, account for potential temporary excess pore pressures during design and include provisions to monitor pore pressure during construction of any structures built over soil deposits subject to consolidation settlement. Monitoring of excess pore pressure has two benefits:

1. Allows control of the rate of load application (the rate of construction), so foundation failure can be prevented.
2. Allows determination of when consolidation settlement has progressed sufficiently to safely proceed with the next phase of construction.

There are several different types of piezometers. The most common and reliable would be the Casagrande piezometer and electronic, vibrating wire piezometers. A Casagrande piezometer is a special type of open standpipe piezometer designed to monitor pore pressure at a specific depth or in a specific soil layer. It is essentially the same as an open standpipe piezometer, with the difference being that the overburden above the sensing area (i.e., the porous tip) is sealed off using a bentonite seal and grout as shown in Figure 7.10.2.2-1.

Alternately, the most common type of electronic piezometer is a vibrating wire piezometer. The porous sensing tip incorporates a vibrating wire strain gauge to measure pore pressure. From original ground surface the installation of a vibrating wire piezometer is the same as a Casagrande piezometer, except a wire comes up from the sensing tip instead of an open pipe.

There are a couple of advantages of using vibrating wire piezometers over Casagrande piezometers. With a Casagrande piezometer, sections of standpipe must be added just as with a settlement plate as the placement of fill progresses. With a vibrating wire piezometer, the wire coming out of the sensing tip is buried in a shallow trench in the original ground surface and taken to a control panel. At the control panel, the wire is either attached to a switching box with the wire from other vibrating wire piezometers or to a data logger. A switching box allows the manual collection of data from multiple vibrating wire piezometers at the required intervals. A data logger will automatically collect data from multiple vibrating wire piezometers at user defined intervals, to be downloaded at the user's convenience. Use of a data logger allows frequent pore pressure observations at regular intervals, at a frequency not practical with manual data collection. For example, pore pressure observations and recordings could be made on an hourly basis, seven days a week, providing comprehensive, uninterrupted data sets from each piezometer. Such data sets would be analogous to high resolution imagery in depicting the current state and progress of consolidation settlement.

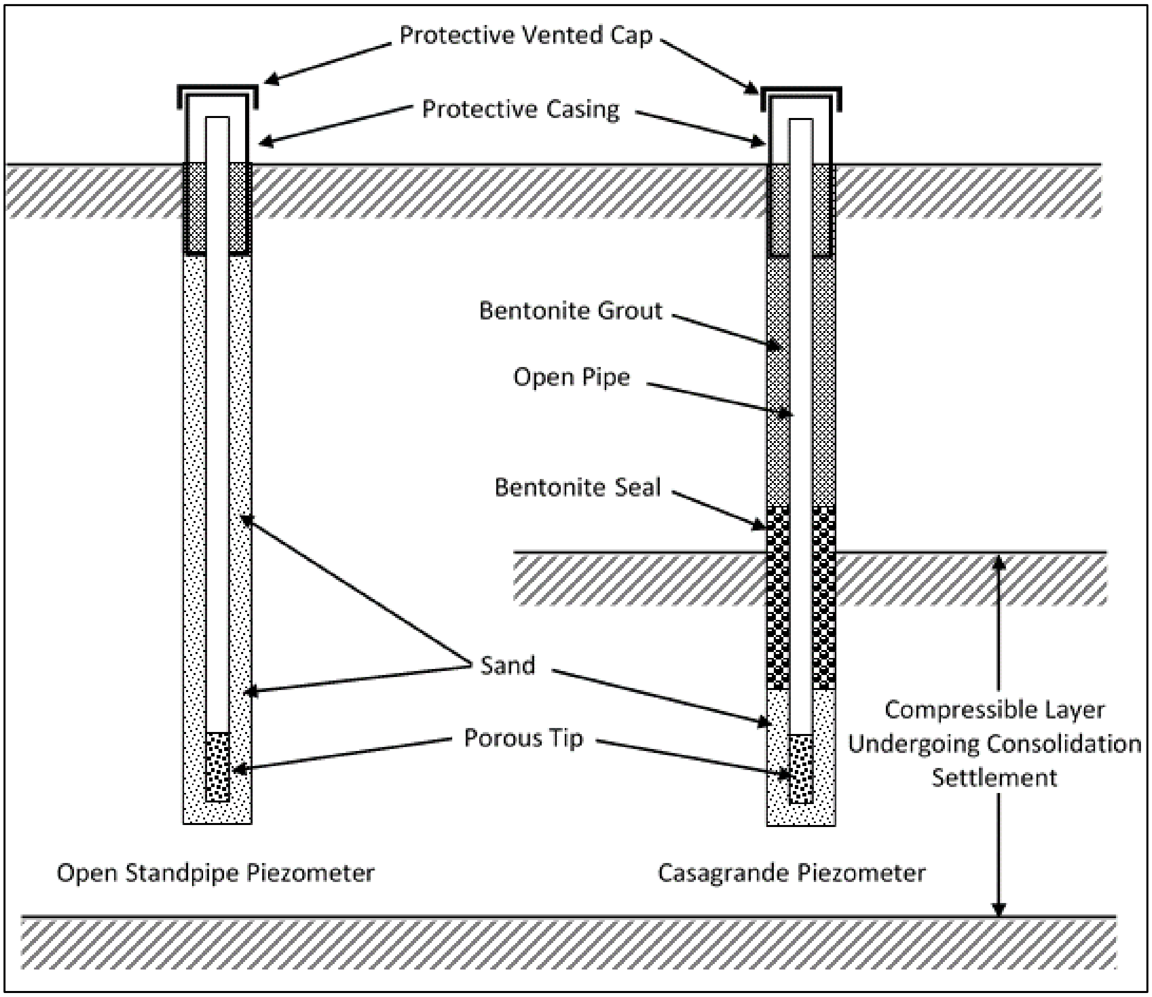


Figure 7.10.2.2-1 – Open Standpipe and Casagrande Piezometers

In terms of an instrumentation plan, the placement of piezometers would be very similar to that of settlement monitoring devices as shown in the example presented in Figure 7.10.2.1-4. The issues discussed and presented concerning redundancy, layout, and survivability for settlement monitoring devices apply equally for piezometers; however, survivability is much less of an issue for vibrating wire piezometers. In addition, the use of a few strategically placed settlement plates is an excellent method for confirming the results of piezometers, adds a layer of redundancy and reliability, and aids in troubleshooting.

An example of such an application is presented in Figure 7.10.2.2-2. This is essentially a rework of Figure 7.10.2.1-4 for settlement plates, showing a similar layout for piezometers, with a select number of settlement plates at concurrent locations. Results from the two devices at the same location provide for redundancy added confidence in both results and recommendations, important for sensitive, high risk and/or difficult conditions.

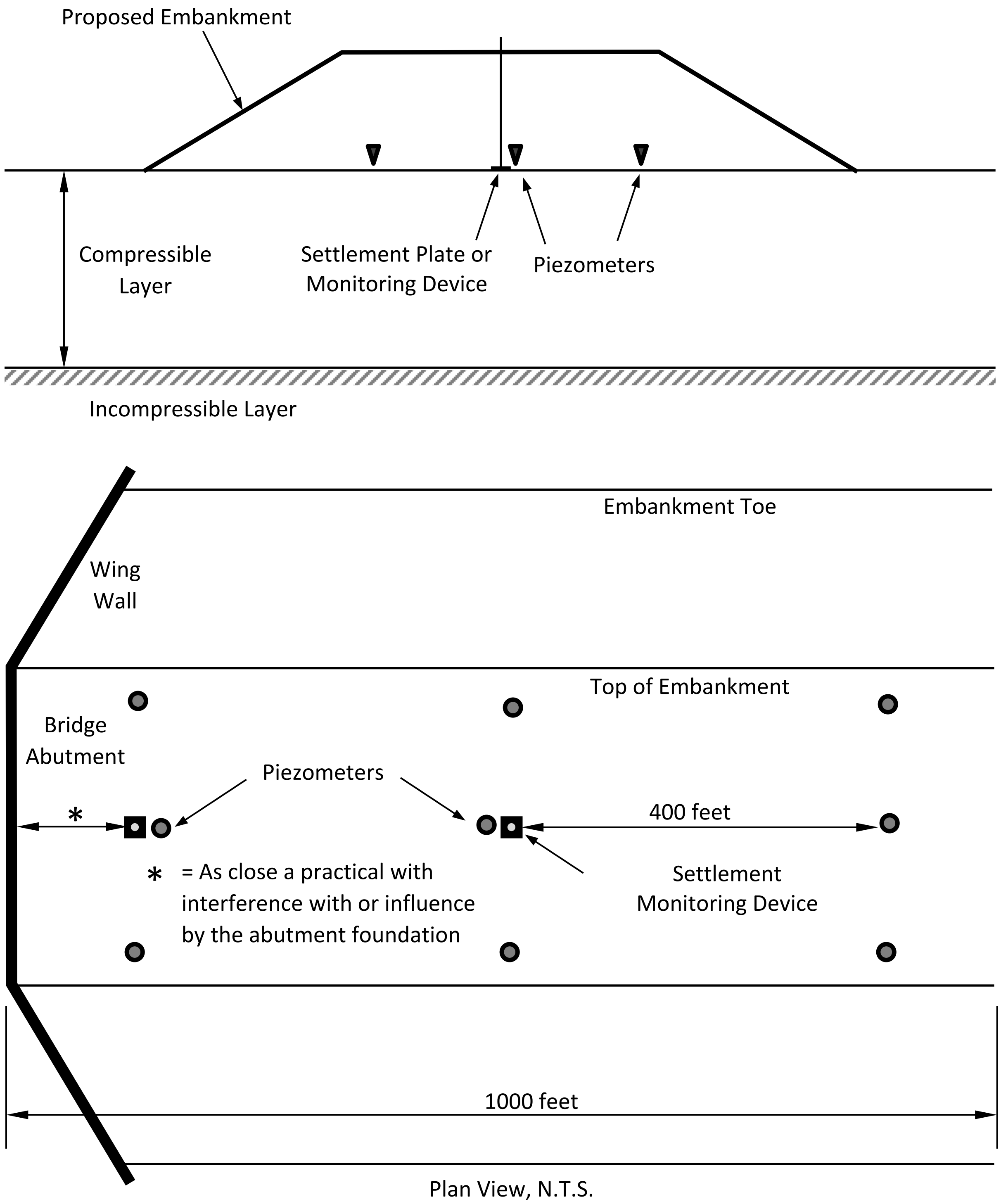


Figure 7.10.2.2-2 – Example Piezometer Layout for Embankment and Structure with Supplemental Settlement Monitoring Devices

7.10.2.3 Other Methods of Settlement Monitoring

There are a variety of other methods of monitoring settlements ranging from simple and inexpensive, to more sophisticated and costly. While covering all these methods is beyond the scope of this chapter, a few other methods will be addressed given their reliability and/or effectiveness.

A straightforward method to monitor settlement is establishing survey monuments on top of an existing structure (e.g., bridge, embankment, culvert, etc.). While this approach is simple and effective, any history of settlement during construction of the structure is lost. Proper placement of survey monuments allows monitoring of differential settlement across structure units, during placement of the superstructure. If only post-construction settlement is of concern, the use of survey monuments can be a simple and inexpensive option. However, even with post-construction settlement monitoring, any available settlement data collected during construction can be very useful in evaluating the long-term settlement effect on a structure. For earthen structures (e.g., embankment, MSE wall, reinforced soil slope, etc.) that are subject to the constant passage of construction equipment, survivability may be an issue, and appropriate protective measures for the monuments must be applied/considered.

There are also several technologically, more sophisticated approaches of settlement monitoring. Systems are available that are highly automated with the ability to data log, provide their own power source, and even upload results to a web site. While technologic sophistication usually comes with an associated increase in cost, a corresponding increase in accuracy and reliability is not always achieved. Therefore, the selection of an appropriate method(s) and/or system of settlement monitoring must satisfy the needs of the project and specific situation. Broadly consider what is necessary to measure and monitor the availability of resources, especially relative to project personnel. Create a monitoring plan that does not place an unnecessary or unreasonable burden on project personnel. Careful attention must be given in preparation of specifications to include adequate performance requirements and standards. This will provide adequate support and backup to project personnel responsible for execution of the monitoring plan. Finally, any settlement monitoring must always consider both risks and returns, while also addressing reliability, cost, and redundancy.

7.10.3 Interpretation of Monitoring Results

When collecting data from settlement monitoring, it is also important that daily records be kept concerning load application. This can be relatively simple for an embankment where tracking and recording the height of fill placed over each settlement monitoring instrument may be sufficient; however, it can be more difficult for irregular shaped structures. With more difficult monitoring, approximations of load placement and date are generally adequate. The data can be used to generate a double y-axis plot versus the date or time plotted on the x-axis. One of the y-axis dimensions will be height of fill or load, with the other y-axis being settlement as shown in Figures 7.10.3-1, 7.10.3-2, and 7.10.3-3).

For any uniform structure (e.g., a retaining wall, etc.) height of fill is typically adequate. As indicated for an embankment, fill height may be sufficient to yield the desired result.

However, because full embankment sections have two sloping sides, fill height relative to instrument location may need to be considered. If it is necessary to locate an instrument within or below the slope face, or near the toe of the slope, load felt in those areas from fill placement may be significantly less than at the center of the embankment section. This must be accounted for in either the interpretation of the results or by adjusting the height of fill placement accordingly. Influence factors (as covered in Section 7.3) may assist in such cases. The other y-axis will either be change in elevation or settlement, or pore pressure depending on whether a settlement plate or piezometer is used, respectively. Examples of results for monitoring of both settlement plates and piezometers are presented in the following figures.

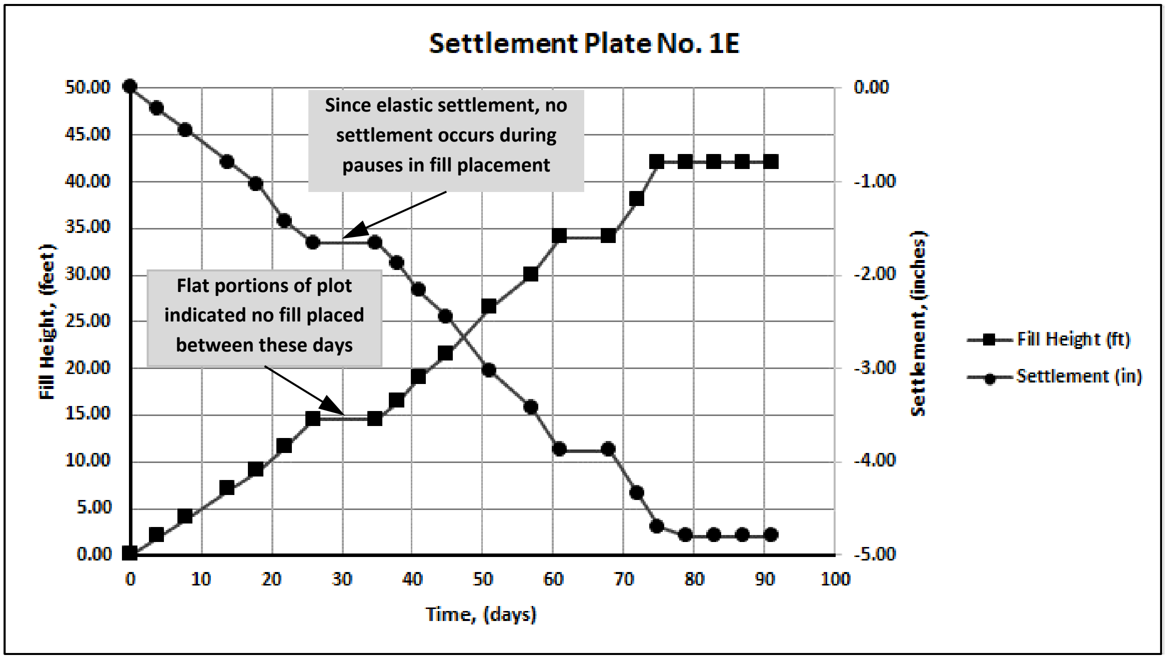


Figure 7.10.3-1 – Time vs Fill Height and Settlement Plot with Settlement Plate Data (Elastic Settlement)

Figure 7.10.3-1 shows settlement monitoring results for a 42-foot-high approach embankment constructed over a 60-foot-thick loose sand deposit. Because the magnitude of settlement was anticipated to be significant, monitoring during construction of the embankment was performed to ensure that settlement was complete before driving piles for the adjacent structure and remove any concern relative to downdrag on the piles. Settlement plates were used to perform the monitoring. The plot indicates time versus both fill height and settlement. Fill height is indicated on the left y-axis and settlement on the right y-axis. Note that settlement occurs as the fill is placed indicating elastic settlement, which is consistent for sand. As the 42-foot-high fill is completed around day 75, the foundation settlement reaches a magnitude of approximately 4.8 inches and is complete. No additional settlement is observed after the completion of the fill and driving of piles can proceed without delay.

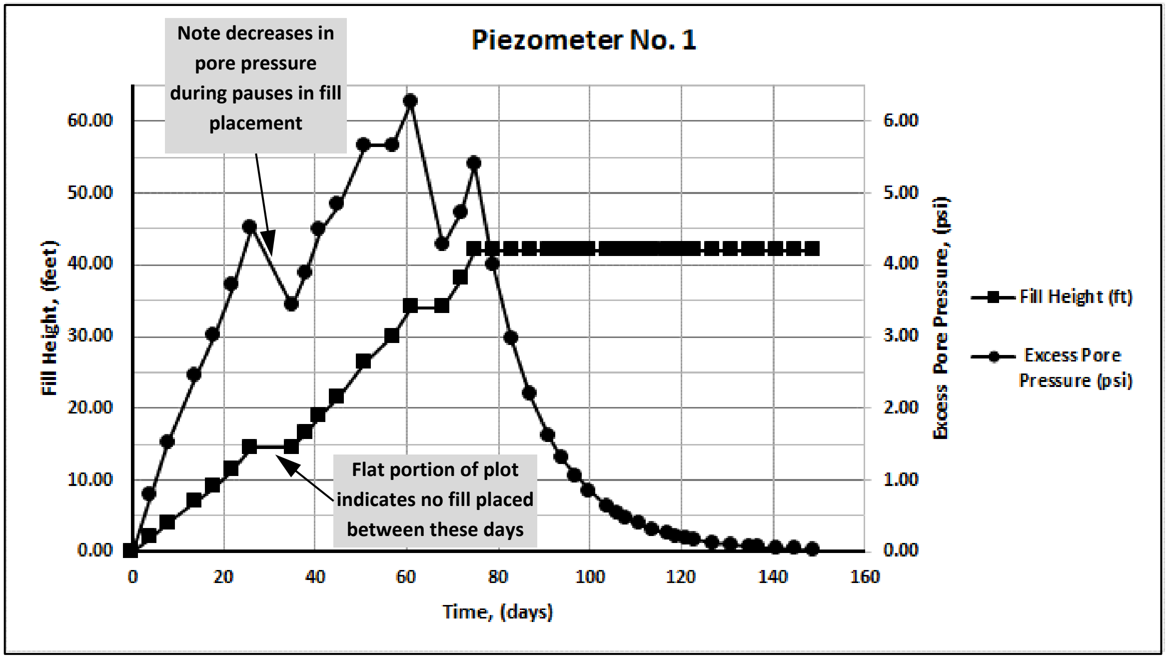


Figure 7.10.3-2 – Time vs Fill Height and Pore Pressure Dissipation Plot with Piezometer Data (Consolidation Settlement)

Figure 7.10.3-2 also shows settlement monitoring results for a 42-foot-high approach embankment. In this case the embankment was constructed over a 60-foot-thick, soft clay deposit. Not only was significant settlement again anticipated, but the time required for settlement was also expected to continue well past completion of embankment construction. Downdrag on piles for the adjacent planned bridge was again of concern. Another concern was that if construction of the embankment proceeds too rapidly, excess pore pressures may increase to a level that could result in foundation failure. Settlement monitoring during construction of the embankment was again deemed prudent to minimize risk and provide protection for both the structure and embankment.

The plot indicates time versus both fill height and excess pore pressure. Fill height is indicated on the left y-axis and excess pore pressure on the right y-axis. Note that excess pore pressures are generated immediately after beginning construction of the embankment, indicating time dependent consolidation settlement consistent with the clay soil present in the embankment foundation. Construction of the 42-foot-high fill is completed around day 75, with foundation settlement continuing as expected. After completion of fill placement, excess pore pressures start to decrease. The rate of decrease is initially rapid, with the decay rate decreasing as the magnitude of excess pore pressure approaches zero. This is consistent with consolidation theory, as the magnitude of excess pore pressure is the hydraulic gradient driving the rate of pore pressure dissipation. As the magnitude of pore pressure decreases, the rate of dissipation decreases.

Excess pore pressures are a direct indicator of both the rate of settlement consolidation and the percent of consolidation that is complete at a given time. As excess pore pressure dissipates, stresses are transferred from pore water pressure to the soil structure, increasing the shear strength of the soil. As pore pressures dissipate and water drains out of the soil structure, there is a volume change resulting in compression of the soil mass and settlement. Therefore, while the pore pressure dissipation curve cannot provide an indication of the magnitude of consolidation settlement, it does directly reflect the rate and state of consolidation.

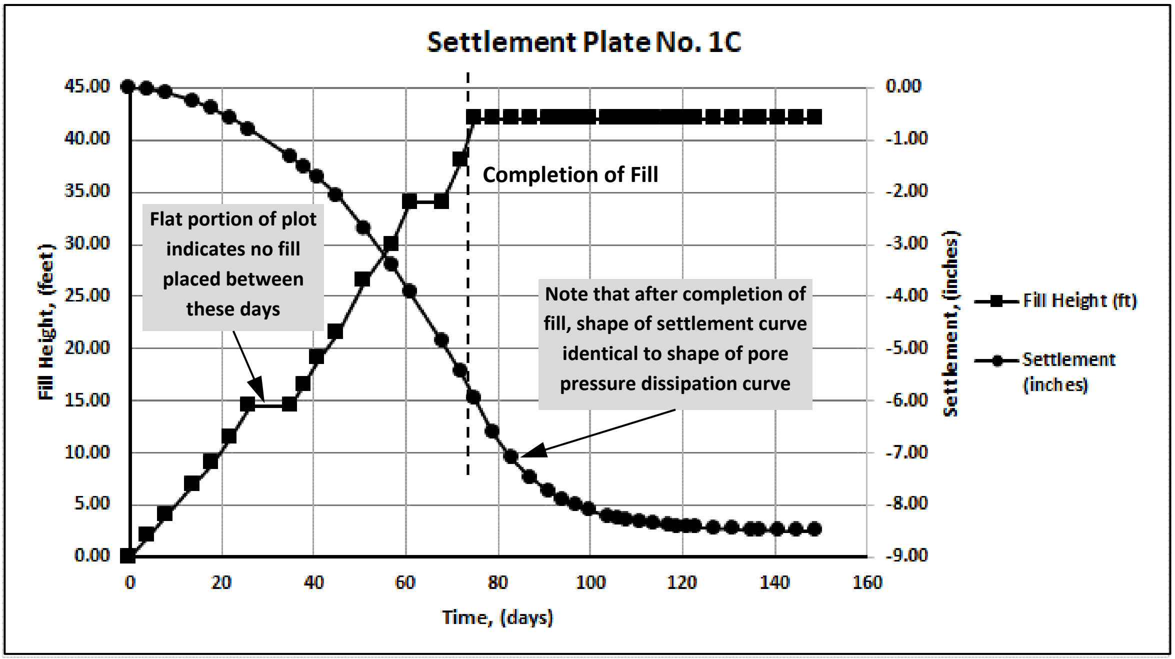


Figure 7.10.3-3 – Time vs Fill Height and Settlement Plot with Settlement Plate Data (Consolidation Settlement)

Figure 7.10.3-3 shows settlement plate monitoring results for the same site as the piezometer results in Figure 7.10.3-2 (i.e., 42-foot-high approach embankment over soft clay deposit). The settlement plate was used to compliment and help validate the collected piezometric data, and to provide a direct measure of the magnitude of consolidation settlement. As can be observed, settlement begins as soon as the load, or fill placement, is applied. As the height of the fill increases, the rate of settlement increases. Once the fill is complete, the rate of settlement begins to decrease consistent with the decrease in the rate of pore pressure dissipation. In fact, if plotted on matching scales, after completion of fill placement on day 75, the settlement curves and pore pressure dissipation curves are identical as shown in Figure 7.10.3-4.

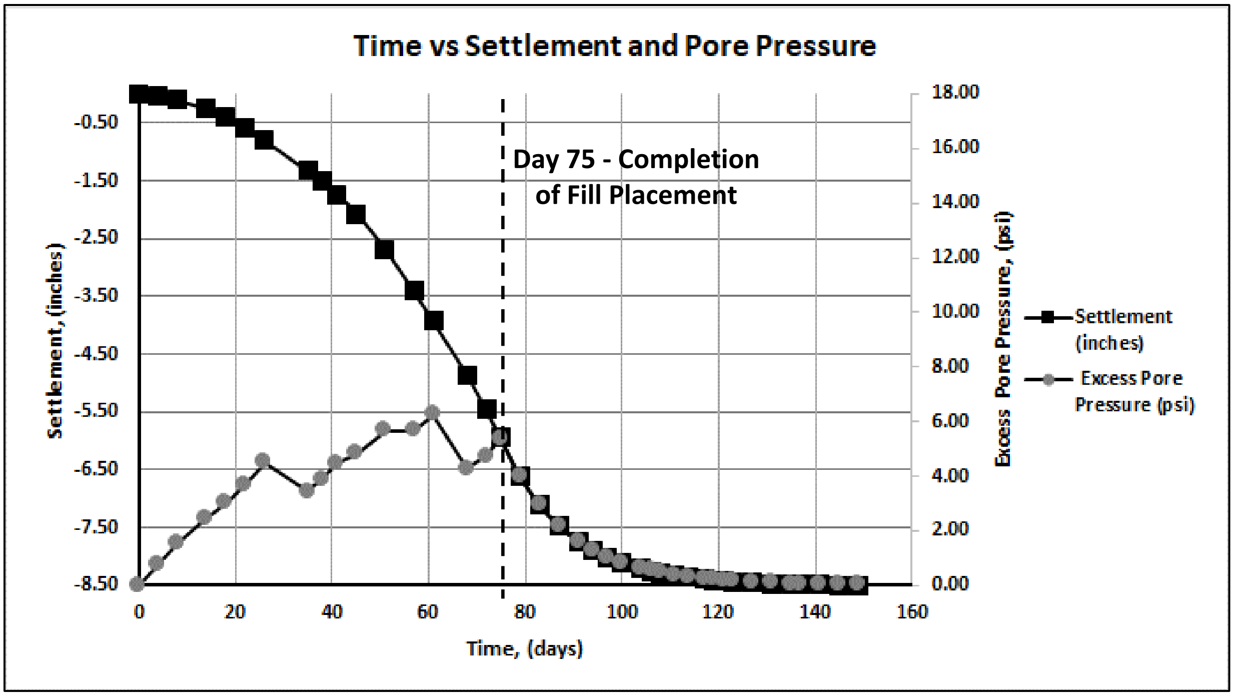


Figure 7.10.3-4 – Plot of Time vs Settlement and Excess Pore Pressure on Matching Scales

7.11 IMPACTS OF SETTLEMENT

When considering the impacts of settlement, consider potential consequences for both the magnitude of settlement and the time-rate of settlement. Complications associated with the magnitude of settlement generally result in some type of physical damage, while issues arising from the time-rate of settlement generally impact contract schedules. Further discussion of these complications is covered in the subsequent sections.

7.11.1 Differential Settlement – Angular Distortion

If uniform settlement of a compressible material below a shallow-founded structure occurs, there are generally limited negative impacts to the structure itself. However, complications may arise due to differential settlement (e.g., if the structure is tied to other existing features, including but not limited to, utilities, existing adjacent structures, or existing adjacent pavement sections). For existing adjacent structures where only a portion of their foundation is impacted, differential settlement within the adjacent structure can result. Any buried structures (e.g., culverts, pipes, etc.) located near or beneath the zone of influence of the current construction activity, may also be impacted.

Non-uniform or differential settlements that occur at varying magnitudes along a structure foundation can be more problematic. Differential settlements induce differential stresses in rigid structure foundations and surrounding components of the structure resulting in problems and/or damage to the foundation and those other components.

At the foundation level, differential settlements can result in cracking of rigid foundation types (e.g., spread footing, slab-on-grade, etc.), tilting of structures, and/or misalignment with adjacent structures or facilities. For rigid structures (e.g., reinforced concrete abutments, walls, etc.), cracks in the footing can easily travel up into vertical abutment or wall components. For abutments, the stresses can further transfer up into the superstructure depending upon when the beam or girder support or connections are placed. In both abutments and walls, severe differential settlement can result in damage to rigid pavements or rigid drainage pipes.

The impacts of differential settlement are generally less severe with flexible structures. Flexible wall systems such as MSE walls are less prone to damage than rigid wall systems; however, if rigid facing panels are used for MSE walls, they can be subject to misalignment or spalling along panel edges or corners. Limit of allowable differential settlement of precast concrete facings and MSE walls is further defined and discussed in DM-4, Section 11.10.4.1. Very flexible structures (e.g., reinforced slopes, soil or rock embankments, etc.) are more capable to distribute stresses induced from differential settlement, which generally help to limit impacts on surface structures such as pavements. However, any structures buried deep and/or located closer to the source of differential settlement (e.g., culverts, etc.) may still be subject to damage or complications.

Potential impacts from differential settlement are highly dependent upon the magnitude of the differential movement, and the lengths over which differential movements occur. In general, the greater the length or distance over which differential settlements occur, the lower the potential for impacts. The quantifiable value used to assess potential impacts from differential settlement is termed “angular distortion.” Angular distortion is simply the ratio of the magnitude of differential settlement to the length or distance over which the differential settlement occurs, expressed in the following equation:

$$\tan\beta = \frac{\Delta s}{L}$$

where,

$\tan\beta$ = Angular Distortion
Δs = Differential Settlement equal to s_A - s_B where
s_A and s_B = Total Settlement at Points A and B respectively
L = Distance Between Points A and B

Requirements for the maximum allowable angular distortion can be found in DM-4, Section 10.5.2.2.

7.11.2 Downdrag

In addition to differential settlement, carefully consider potential downdrag forces from settlement when using deep foundations that extend through compressible material subjected to additional loading (e.g., fill embankments, etc.) or deep foundations that extend through embankment material subject to settlement under self-weight (e.g., high embankments constructed of rockfill, etc.). Deep foundation types, such as driven piles in particular, installed through a compressible material can develop negative shaft resistance, also termed downdrag, along the length of the piles. Account for the effects of downdrag in the design following the requirements of DM-4. If the downdrag force is of sufficient magnitude, the reduction of the frictional resistance along the length of the pile can result in damage to the piles and the supported structure. Figure 7.11.1-1 illustrates the mechanism of downdrag and its impact.

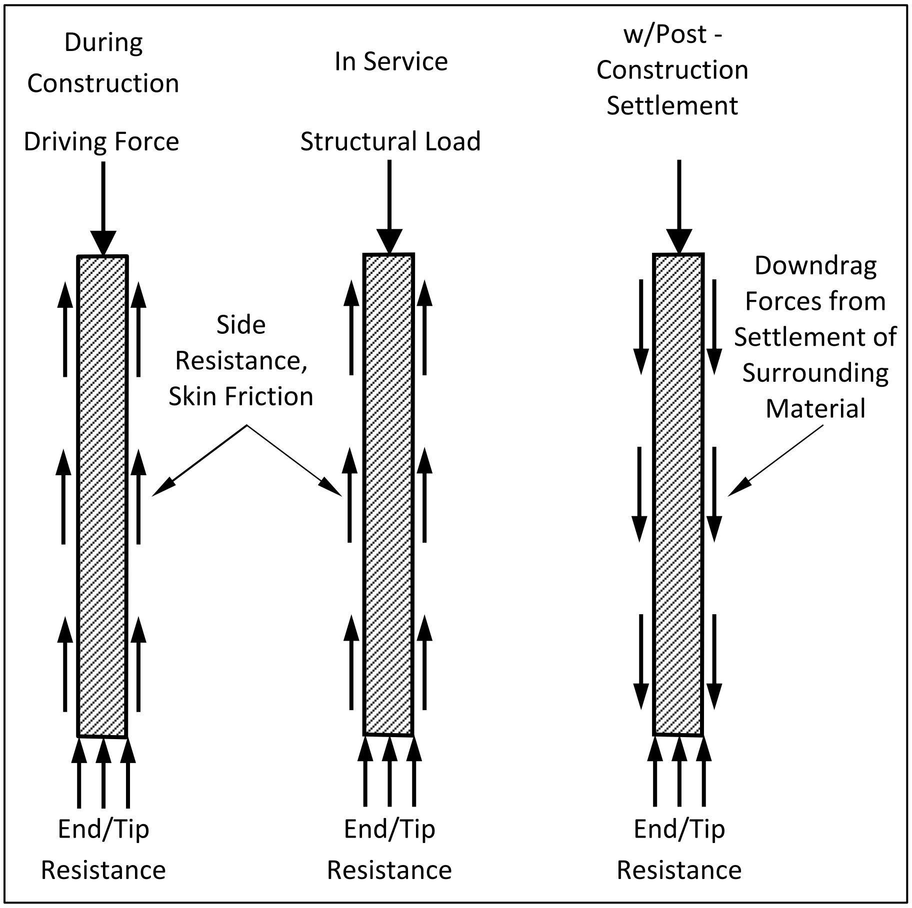


Figure 7.11.1-1 – Downdrag on Piles Due to Post-Construction Settlement

During driving, dynamic loads are applied to the pile and shaft and tip resistance is generated along the side and the tip of the pile, respectively. Designed and constructed properly, the driving force/resistance applied to the pile during installation is the highest loading the pile will experience, which inherently exceeds service load conditions. However, if the downdrag force reaches sufficient magnitude given settlement of the material surrounding the pile, the resistance acting along the length of the pile will be reduced until the downdrag force is so great that the shaft resistance is completely lost.

This loss in the shaft resistance means that the pile must resist the applied structural loads (i.e., controlling strength limit state), and now also the induced downdrag force. Therefore, the pile is subjected to resisting additional downdrag load that it was never intended, or designed, to resist. If the increased loading exceeds the resistance of the pile, failure including buckling, crushing, or vertical displacement of the pile can occur. AASHTO LRFD 3.11.8 provides a step-by-step procedure on how to evaluate force effects due to downdrag on deep foundation components. Methods of mitigating pile downdrag are discussed in Section 7.12.

Consequences resulting from excessive settlement that was not adequately addressed during design can be substantial. The impacts can range significantly and can compound on one another. These impacts may include:

- Damage to new or existing structures or facilities
- Time and costs to repair or replace damaged components
- Reductions in service life
- Time and costs for mitigation to prevent additional damages
- Project delays and disruption of construction schedules
- Construction claims as the result of project delays
- Highway user lost time and associated costs

Considering the range and potential magnitude of the identified consequences, sufficiently investigate and address all potential sources of settlement for proposed construction during design. Adequate construction provisions must be prepared and included in the contract and plans to prevent or mitigate complications.

7.11.3 Impacts of Settlement – Time-Rate

Complications associated with the time-rate of settlement can have both physical and contractual impacts. If adequate provisions are not prepared and included in contract documents, or if time dependent settlements were simply not anticipated on a project, issues and conflicts concerning construction schedules and a contractor's planned operations often arise. Provisions must be prepared to effectively monitor and measure both the magnitude and rate of consolidation settlement. This not only includes an instrumentation plan, but also a special provision detailing an action plan based upon the results of the observations. It is critical that the action plan contains language and requirements clearly defining when work can proceed on any contract construction components that may be negatively impacted by consolidation settlements.

Such provisional requirements provide the Department's representative on a project the contractual capability to control the contractor's operations relative to construction activities that may be negatively impacted by settlement, without incurring any contractual liability. Put simply, it prevents the contactor from submitting a legitimate construction delay claim. The provisions also protect any structures and components that may be negatively impacted or damaged if construction could proceed before completion of settlement.

Include language and requirements in the provisions to allow the Department's representative control over the rate of construction, or the rate of load application on the compressible layer, to prevent possible damage or failure of the foundation. Regular and frequent observation of excess pore pressures allow evaluation of the foundation's and compressible layer's current stress condition, so that loads are not placed more rapidly than the time required for the excess pore pressures to dissipate.

In addition to protecting the Department, the provisions also protect the contractor. The contractor can bid the contract fully informed of any restrictions that may impact schedule or operations, so that a fair and equitable bid can be prepared. It also allows the contractor to bid competitively against all other prospective bidders, with the full knowledge that all other bidders will have to meet and abide by the same requirements.

Failure to adequately address the specific needs of time dependent consolidation settlements during design and preparation of contract documents (e.g., plans, specifications, provisions, etc.), can result in serious and costly complications. Potential consequences include:

- Damage to embankments, fills and their foundations
- Embankment and fill slope failures
- Time and costs to repair or remediate failed fill slopes
- Time and costs of mitigation to prevent additional damages
- Project delays and disruption of construction schedules
- Unnecessary and costly construction claims as the result not providing adequate construction control requirements
- Highway user lost time and associated costs

As with settlement magnitude, considering the impacts and costs of the potential consequences. Time-rate of settlement for proposed construction must be adequately investigated and addressed during design.

7.12 MITIGATING AND EXPEDITING SETTLEMENT

When anticipated or observed settlements exceed design values or serviceability limits, there are several approaches available to address the specific situation. Consideration of the appropriate approach depends upon a multitude of variables, including:

- Type of Settlement (i.e., elastic vs consolidation)
- Materials present and subsurface conditions (e.g., non-plastic granular, non-plastic fine grained, plastic fine grained, mixed grained, depth and layering of soil strata, saturated vs non-saturated, geologic conditions, etc.)
- Site Conditions and Constraints (e.g., accessibility, adjacent structures, adjacent traffic, nearby waterways, etc.)
- Type of Construction (e.g., embankment, bridge, wall, culvert, settlement of adjacent roadway, etc.)
- Risks (e.g., options relative to implementation, performance and outcome, impact on adjacent structure and facilities, traffic safety, complicating factors, etc.)
- Reliability (i.e., expectations for success of various options)
- Cost (i.e., most economical option, cost vs benefit)

As can be observed, there are a wide range of factors that may impact the selected approach or mitigation of a potential or active settlement problem.

7.12.1 Mitigating and Accelerating Anticipated Settlement

The following discussion relative to addressing settlement is typically more applicable to time dependent consolidation settlements because elastic settlements occur as load is applied, and usually either do not present a problem or are much less difficult to manage. However, in certain circumstances, mitigative options could be applied to situations involving elastic settlements. When broadly addressing concerns with settlement for a given set of conditions, two methods include prevention or avoidance of potential settlement, and management of potential settlement. These methods can be further broken down into specific mitigative options as presented in the flow chart in Figure 7.12.1-1. The following discussion will refer to the options listed in this figure. The mitigative options presented in the flow chart are elaborated upon in the subsections below. Some overlap will likely be apparent in discussion of these generic concepts. Consider that all mitigative options are likely not available or practical for every situation and/or set of conditions. Job specific needs, constraints, nature of the construction, subsurface conditions, general site conditions, and a range of other factors may all impact or dictate what options are viable.

7.12.1.1 Prevention or Avoidance of Potential Settlement

Settlement can be mitigated by preventing or avoiding settlement entirely. Settlement can be avoided by rerouting the proposed construction or selecting a design option that does not impact the compressible layer. As shown in Option 1.1a of Figure 7.12.1-1, rerouting the proposed construction is nothing more than avoiding the area with the compressible layer of concern. For this option to be viable, the compressible layer would likely have to be very limited in area and lateral extent. Realignment may require additional right-of-way or may have consequences impacting multiple other elements of proposed construction. In such cases, realignment may only be warranted if the expenses and impacts of a realignment option is more economical than mitigating the anticipated settlement. In rare instances; however, rerouting proposed construction may be the best or only option in situations where the cost of settlement mitigation by other methods are so prohibitively high, as to render them impractical.

As shown in Option 1.1b, another possibility to avoid settlement would be to select a design alternative that does not impact the compressible layer by spanning the compressible layer of concern, avoiding any impact and possible settlement. The foundation of the spanning structure would either have to be constructed through the compressible layer transferring stresses to a suitable stratum, or be sufficiently outside the zone of influence of the compressible layer so that the resulting stress increase and settlement magnitude would be acceptable or manageable. Unlike the realignment option, there will be little impact on other items of proposed construction and essentially no impact on right of way. As with the realignment option, spanning the compressible layer may only be warranted if the expense of a spanning structure is more economical than mitigating settlement. However, the spanning option may be more cost effective than realignment if mitigation is cost prohibitive.

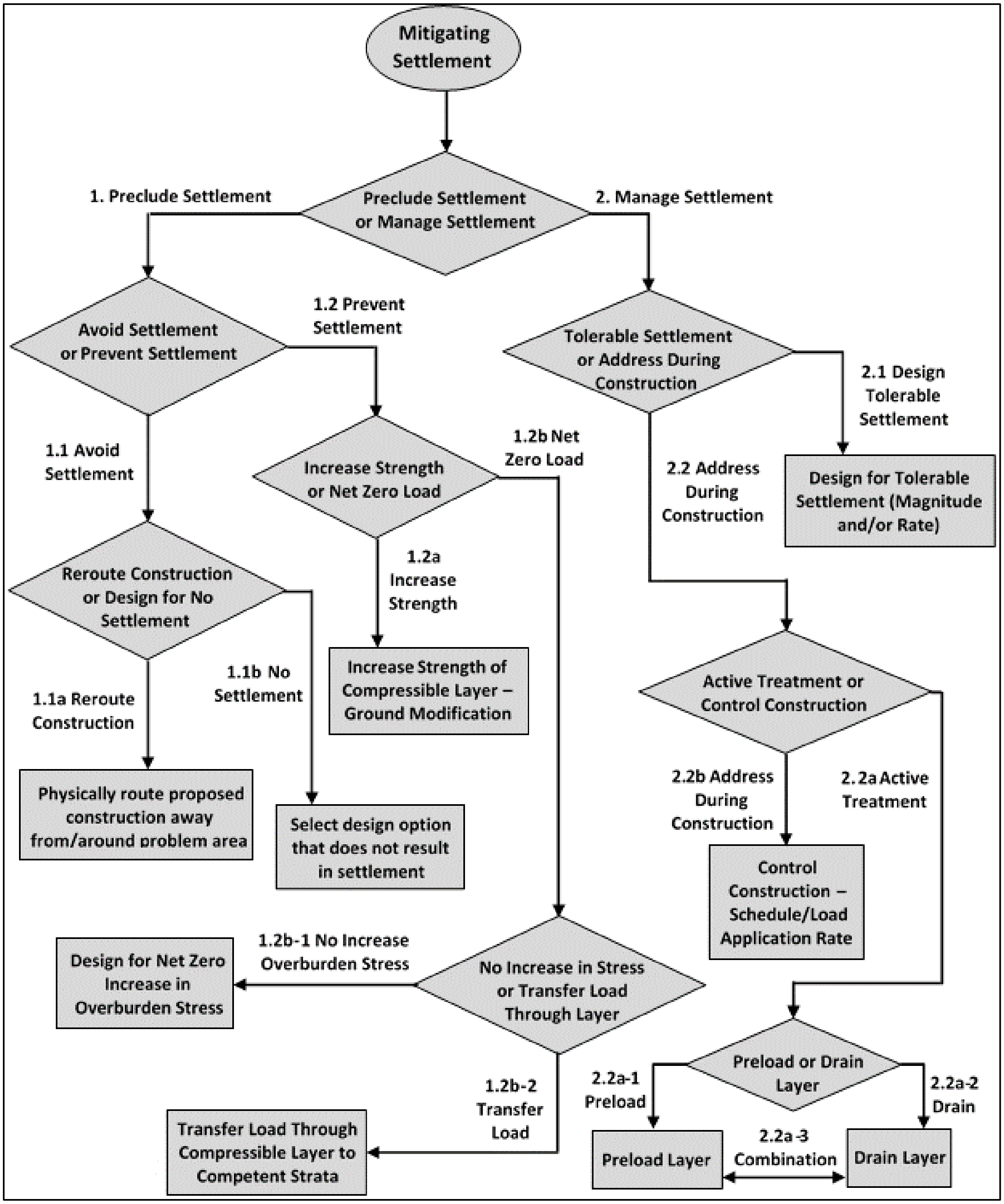


Figure 7.12.1-1 –Addressing Anticipated Settlement of a Compressible Layer

Potential settlement can also be prevented by ground modification, load reduction, or a transfer of load through the compressible layer. If Option 1.2a was selected as best practice, some type of ground modification technique to improve the strength and density of the compressible layer would typically be needed. There are a variety of ground modification methods with the approaches applicable to this discussion generally involving soil displacement. These techniques include compaction or low mobility grouting, stone columns, and displacement piles. Soil mixing and jet grouting are also ground modification techniques that strengthen the soil mass by adding and mixing cement into the soil but are more often intended to transfer applied loads through the weak or compressible layer to a more competent stratum. Displacement methods increase soil strength by densifying the surrounding material. More discussion on densification and compaction is discussed in Chapter 9 of this publication. More discussion on Limited Mobility Grouting can be found in Appendix C. These methods are analogous to preloading a compressible layer, except instead of a vertical surcharge being applied over top of the compressible layer, the mass is loaded internally with radially applied stress.

Another alternative (Option 1.2b-1 of Figure 7.12.1-1) to prevent settlement would be to reduce the load resulting in a net zero change (i.e., no change) in overburden stress. With no change in vertical stress on the compressible layer, there can be no void change and compression. This condition can be accomplished by load balancing with the use of lightweight fill materials. Any increase in overburden stress from the placement of fill is compensated with removal of adequate existing materials and/or replaced with lightweight fill materials. The result is zero net change in overburden stress in the compressible layer. This option may be particularly useful for vertical structures with a limited footprint (e.g., retaining walls, culverts, abutments, MSE walls, etc.). Reference Chapter 13 for use guidelines of Geofoam.

As previously mentioned as an option to avoid settlement via spanning the compressible layer, transfer of the load through the compressible layer can prevent settlement. This alternative (Option 1.2b-2) involves simply transferring applied loads through the compressible layer. Piles or other vertical members are driven or drilled to a competent load bearing material, carrying any applied loads to the more competent material. In addition to driven piles, additional deep foundation components such as predrilled piles, drilled shafts, soil mixing, and jet grout columns are other options for the vertical load transfer members. When using vertical load transfer members to transfer load through a compressible layer, an intermediate structure will be required to effectively transfer the applied load to the piles. This may be in the form of either a pile cap for a rigid structure with driven piles or drilled shafts, or a flexible load distribution pad for a spread footing, culvert, or flexible structures (e.g., embankments, reinforced slopes, MSE walls, etc.).

Complimentary to Option 1.2b-2 and as discussed in Section 7.11.1, the effects of negative shaft resistance and resulting downdrag are to be considered and addressed when deep foundation components extend through compressible material or material susceptible to settlement. In instances where the calculated negative shaft resistance/downdrag is substantial and where accounting for the reduction in resistance would be impractical for the design, DM-4, Section 10.7.1.6.2, recommends the use of casing to mitigate the effects. The intent of the casing is to prevent direct contact between the soil and deep foundation element above and through the zone susceptible to settlement. Use of bitumen coating, or another viscous coating, applied to deep foundation elements is also discussed, but the effectiveness of the coating is unreliable given the unlikely survivability of the applied product post installation; therefore, it is not recommended for use.

7.12.1.2 Manage Settlement

Mitigating settlement can also be managed on a project. Settlement can be managed by designing for tolerable settlement. The approach of Option 2.1 is to produce a design where the settlements induced from the application of loads can be tolerated for the type of proposed construction. This may involve the total settlement anticipated or only the remaining anticipated settlement after the completion of a specific component or phase of construction.

Settlement can also be managed by preloading the compressible layer, draining the compressible layer, or combining the two options to address consolidation settlement of the plastic compressible soil layers more effectively. Option 2.2a-1 of Figure 7.12.1-1, suggests preloading the compressible layer to manage the settlement. Preloading usually consists of placing a specified load of fill over the compressible layer that increases the overburden pressure, providing means for volume change and compression, before the start of the proposed construction. By preloading an area, settlement is allowed to progress to an appropriate level such that any additional compression will not have an adverse impact on the proposed construction. At that point, subsequent construction activities are allowed to proceed in the area.

Because the magnitude and cost of placing fill is significant, preloaded fills are often planned as part of the permanent construction. These include highway structures planned over compressible layers (e.g., embankments, approach embankments to bridges, side-hill fills, etc.). Surcharge loading, or additional fill placement beyond what is required for final conditions, is frequently done to ensure adequate settlement is achieved for the final construction condition. Once the surcharge fill is removed after the required level of consolidation is achieved, the overburden pressures from the final construction condition (e.g., subbase, pavement, anticipated future loads, etc.), remain below that which has been already applied during preloading. Surcharge loads can have other benefits that are discussed in more detail in Section 7.12.2.

Option 2.2a-2 suggests draining the compressible layer. Draining the compressible layer usually consists of drilling or inserting a series of vertical drains extending into in the compressible layer. The drains may be prefabricated or consist of a sand column. As discussed previously, the time required for consolidation settlement to occur is a function of the compressible soil layer permeability, k, and the length of the longest drainage path squared, H_d^2. An example is provided in the Settlement – Worked Examples webpage.

Furthermore, as noted in Option 2.2b, settlement can be managed through contract provisions to control construction rate and/or sequence. Situations develop where it becomes necessary to contractually control the activities of the contractor without risk of incurring a legitimate construction delay claim. When dealing with consolidation settlement, consider the need for contract provisions allowing this control or limitations of contractor operations. One such circumstance involves consolidation settlement of compressible layers during construction, and whether the need to control the rate of load application and/or the need for fill quarantine periods is required. Also, for very low permeable, compressible layers, excessive pore pressure increases may occur if loads are applied too rapidly, even if additional measures are taken to accelerate settlement such as enhancing the drainage. In such instances, controlling the rate of load application by monitoring and evaluating the soil pore pressures will be necessary to preclude foundation failure and resulting slope failure of the embankment.

As was discussed in Section 7.7, pore pressure (u) has a significant impact on soil shear strength. There is no specific excess pore pressure value or ratio of excess pore pressure relative to fill height at which conditions become critical and failure is imminent. Determination of critical excess pore pressure requires analysis using site specific subsurface conditions, geometry and loads at various stages of construction. In addition to analysis for long-term conditions, conduct analyses at intervals of 20% of the maximum embankment height up to the maximum embankment height, including any temporary surcharge, to assess critical states of excess pore pressure in the compressible layer. Based upon the results of the analyses, appropriate controls for excess pore pressure can be established and included in the contract special provisions.

If after completion of fill placement, the presence and active dissipation of excess pore pressures still exist in the compressible clay layer, this indicates that consolidation settlement is not yet complete. A knowledge of hydrostatic pressure before placement of fill allows reliable assessment of percent of consolidation that has been completed. When monitoring excess pore pressures and settlement during construction conditions, the required percent completion of consolidation and/or maximum magnitude of settlement over time (settlement rate) are established in the contract special provisions. The assessment of percent consolidation complete, along with observation of excess pore pressure decay and settlement rate, determines when the contract special provision requirements have been met and the next phase of construction can proceed.

The individually presented mitigative options may also be used in conjunction with one another such as development of contract special provisions to control the construction rate (Option 2.2b), combined with preloading (Option 2.2a-1), and draining the compressible layer (Option 2.2a-2), before the start of construction. While some mitigative options lend themselves specifically to addressing consolidation settlement, others can be applied more generally.

7.12.2 Surcharge Loads

The magnitude of settlement at any given percentage of consolidation, is completely dependent upon the magnitude of the applied overburden pressure. The higher the load, the greater the magnitude of settlement at any given percentage of consolidation. While the rate of pore water drainage controls the rate of consolidation, the magnitude of the applied load controls the magnitude of settlement. Therefore, by applying a greater surcharge load, more void reduction/soil compression and greater total settlement, can be achieved in the same amount of time. Figure 7.12.2-1 illustrates how increasing the surcharge load, while not increasing the rate of consolidation, can significantly reduce the time to reach a required magnitude of total settlement.

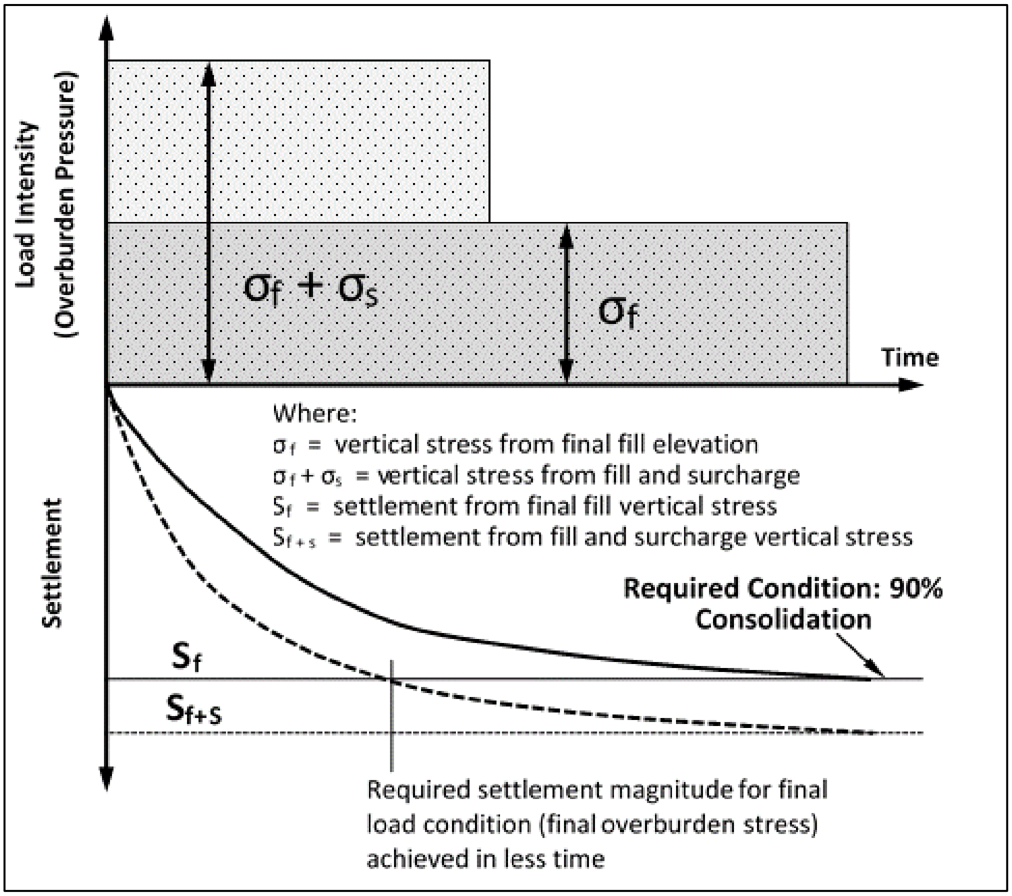


Figure 7.12.2-1 – Impact of Surcharge on Consolidation Settlement

Referring to the example presented in the Settlement – Worked Examples webpage, following Option 1.2b-2 of Figure 7.12.1-1, implementing the use of vertical drains is a way to expedite the time rate of consolidation given that the drainage path length is a function of the vertical drain spacing. In summary, implementing these mitigative efforts ensures that settlement from post-construction loads is precluded.

7.13 SETTLEMENT OF FILL AND FILL EMBANKMENTS

If constructed fill embankments settle, a reduction in serviceability of a structure or pavement may occur. The source of settlement may either be with the fill material itself (e.g., use of unsuitable material not meeting specification, improper placement/compaction, etc.) or with the foundation materials below the fill embankment. Generally, issues that affect the proper placement of fill material are moisture content outside the optimal range (i.e., too wet or too dry), excessive lift thickness, improper selection or use of compaction equipment, lack of construction oversight, or some combination of these conditions. In instances where the foundation material is contributing to the settlement, the cause of deformation may be due to the presence of unforeseen conditions not discovered during the subsurface exploration and subsequently not accounted for in the design. Regardless of the source, and depending upon the severity of the deformation, corrective measures must be developed and implemented.

7.13.1 Considerations for Structure Backfill

Settlement at bridge approaches is a problem often referred to as "the bump at the end of the bridge." Three factors known to have a direct impact on settlement at bridge approaches are deformation within the structure backfill, deformation of the foundation soil(s) below the structure backfill, and drainage.

Deformations within structure backfill may be the result of improper compactive effort, volumetric changes (i.e., post-construction densification induced from traffic vibrations), post-construction settlement of shallow foundations, and bearing capacity failure beneath the sleeper slab. When compacting structure backfill, reference the Standard Drawings. Use of geosynthetics within the bridge approach provides additional reinforcement as discussed in a case study by the Tennessee Department of Transportation (TDOT), titled Geosynthetic Reinforced Fill Material for Tennessee Bridge Approach Slab Support. Use Publication 72M, RC-15M, Geosynthetic Stabilized Bridge Approach (GSBA), for the construction of geosynthetically stabilized bridge approaches.

Failure of the foundation soils below the structure backfill can also result in vertical and horizontal deformations at the bridge approach. If site conditions warrant, evaluate failure modes including, but not limited to, the following:

- lateral squeeze (resulting in an additional force acting on deep foundation components)
- post-construction foundation settlement
- global stability of shallow foundations.

Drainage is the final factor known to contribute to bridge approach settlement. Failure modes triggered by improperly designed or malfunctioning drainage may include, but are not limited to, the following:

- surficial slope erosion (e.g., lack of slope armoring, outlet placement, etc.)
- abutment slope instability (e.g., eroded soil/rock placed in front of the abutment reducing resisting force, etc.)
- increased hydrostatic pressure (e.g., failed/clogged drainage behind structures and increasing the lateral force, etc.)
- lack of drainage below approach slabs (e.g., ponded/trapped water is susceptible to volumetric changes from freeze/thaw, etc.)

Beyond specifying field inspection continuously throughout construction, field instrumentation can be specified to monitor the performance after construction is completed and during the life of the structure, as well as to evaluate the cause of any deformation observed over time. Field instrumentation can include, but is not limited to, survey monuments, settlement plates, inclinometers, and piezometers.

7.13.2 Considerations for High Rockfill Embankments

Statewide, case studies have been conducted on the settlement of large rockfill embankments ranging in heights from 25 feet to 160 feet. Many people have a false belief that if properly placed, rockfills are subject only to short term elastic settlement and that the rockfill embankment itself does not settle. However, rockfill embankments can and do experience post-construction settlement. While the magnitudes of post-construction settlement are typically not as significant as those often observed for soils, the consequences can be equally severe. There is no best practice for calculating settlement of a rockfill embankment. Rather, the estimated settlement for a rockfill embankment can be lessened by understanding the risk involved with the various components and practices employed in creating the rock embankment, and selecting materials and construction practices to meet the desired outcome for project settlement in the time available.

Sources contributing to the internal deformation of large rockfill embankments are dictated by environmental conditions (e.g., changes in hydrostatic conditions, rainfall events, etc.), material characteristics (e.g., rockfill gradation, rock durability, etc.), and procedural operations (e.g., construction controls, monitoring, installation procedures, etc.). A few hypothetical examples include:

- Construction is performed during a dry period. This results in a false indication that internal settlements have completed, only to become active when normal rainfall conditions return.
- Weaker and lower quality rock materials are placed in higher rockfills. These rock materials are subject to deformation and breakdown over time at stress concentration points that leads to post-construction volumetric deformation resulting in internal settlement of the rockfill.
- Poor material control, construction control, and/or placement practices are observed during the project. This results in poor compaction contributing to post-construction internal settlement.

Details of the identified sources and ways to mitigate the effects of settlement within rockfill will be discussed below. This discussion does not account for settlement of the foundation material; however, if the foundation material is susceptible to settlement, that must be factored into the final design and monitoring of the proposed rockfill embankment.

High embankments constructed of rockfill warrant special consideration given their size and stress response (i.e., susceptibility to hydrocompression and crushing). Hydrocompression occurs when rockfill gets wet and the additional moisture causes the finer grained components to become buoyant, realign/weaken, and then get crushed at contact points, resulting in a reduction in volume (i.e., internal settlement of the rockfill embankment). Regarding embankment size, the pressure at the base of a 100-foot-high rockfill embankment could be in the range of 95 psi to 100 psi. When constructing rockfill embankments of that magnitude, consider the potential for fracturing or crushing within the rockfill, particularly if very large uniform particle sizes are used. Assuring good rock quality and durability is important in such situations. This is not to say that internal settlement and crushing only occurs in rockfill embankments with heights exceeding 100 feet.

The large particle size typical of rockfill makes it one of the most difficult materials to compact. During the compaction process, the effort generally results in particle organization as opposed to densification of the material. If weaker rockfill materials are present, they may undergo some breakdown during the compaction process. While some limited breakdown is generally beneficial as breakage fills open voids, non-durable rock must not be specified in critical areas (e.g., under foundation components, etc.), unless the non-durable rock material is broken down and compacted as a soil. Any analysis modelling the embankment must assume soil design strength parameters.

In areas known to have degradable rock, conduct consistent visual classifications in the field, supplemented by slake testing. Special provisions must guide construction activities relative to settlement expectations based upon evaluation of the classifications and test results. Rockfill and durability requirements are detailed in Publication 408, Section 206 and Section 850, respectively. As previously mentioned, even when more durable rockfill material is placed and the particles remain locked during compaction, if done so during an exceptionally dry period, any sudden significant changes in groundwater conditions or rainfall can result in weakening and lubrication of contact points, resulting in relatively rapid and potentially significant internal settlement of the rockfill. Therefore, to mitigate excessive and/or extended post-construction settlement, it is critical that the rockfill is adequately wetted during placement and compaction to avoid hydrocompression during rainfall events or other changes in groundwater conditions.

When rockfill embankments are built to support structural components, the benefits and limitations of both shallow and deep foundation options are to be considered as the estimation of the internal settlement of rockfill is subjective. Approach the use of shallow foundations on rockfill embankment with great caution as there is a history of Department projects where the actual amount and rate of settlement exceeded the design estimated settlement values. This is partially due to the past practice of assuming that the internal settlement of the rockfill embankment does not occur or is assumed to be marginal.

7.13.2.1 Foundation Type and Best Practices

As stated previously, a best practice/methodology to compute settlement of the rockfill embankment is not currently available. While the use of deep foundations with shallow foundations on rockfill for the same structure is possible, it is the Department’s preference to keep the same foundation type across each substructure unit founded on the same rockfill embankment, given the subjective nature of the rockfill settlement estimates. However, certain foundation types may prove more practical for certain site-specific conditions, even within the limits of the same structure. In those instances, use of different foundation types within the same rockfill embankment requires approval from the CGE. Designers may follow one of two approaches in regard to founding structures on rockfill embankments:

1. **Support the structure on deep foundations through rockfill embankment.** For deep foundations, such as driven piles, downdrag forces caused by settlement of the rockfill material surrounding the installed piles must be accounted for in the design and as specified in DM-4. A pile driving window is to be constructed within the rockfill embankment to facilitate pile driving activities by limiting obstructions and to ensure that piles are driven to the specified tolerances. The designer must use sound engineering judgement when estimating the amount of settlement and resulting downdrag force anticipated for the deep foundations. Currently, no accepted standard approach or state of practice exists for deep foundations extending through rockfill embankments susceptible to settlement under self-weight. Therefore, a conservative estimate is advised. Further discussion on downdrag is available in Section 7.11.1, and discussion of ways to mitigate/eliminate the concern for downdrag are discussed in Section 7.12.1.1, Option 1.2b-2.

   Note: Future revisions of Publication 408, Section 206, will specify the use of a pile window when driven piles are to be installed through a rockfill embankment. The pile window zone is to be constructed out of aggregate having greater than 70% of the material passing the 3/8-inch sieve (less than 30% retained on the 3/8-inch sieve) and less than 20% passing the No. 200 sieve. This gradation is such that it will not impact pile driving activities and will allow the compacted material to be tested for in-situ water content and density. Consider the use of a geotextile to separate the coarser rockfill materials from the aggregate within the pile window zone. A Standard Drawing detailing the construction of a pile window within a rockfill embankment is currently under development and will be incorporated into Publication 219M, Bridge Construction Standard Drawings, upon completion.

2. **Support the structure on a shallow foundation on rockfill embankment.** For shallow foundations constructed on rockfill, geosynthetics can be specified to increase stability of an embankment by increasing the strength/reinforcement or by preventing the migration/separation of fines. Use of a geosynthetically reinforced zone immediately below a shallow foundation on rockfill may prove beneficial as it could reduce the effects of settlement and potentially increase bearing resistance. Additionally, wrapping a separation geotextile around the reinforced zone could also increase stability by separating the specified material within the reinforced zone from the larger material present within the rockfill embankment. Given the inherent variability of onsite borrow material, its use for backfilling the reinforced zone may prove difficult in controlling density and moisture in the field. If adequate construction controls cannot be put into place, a deep foundation type is recommended.

   Past practice for Department projects where shallow foundations were proposed with successfully preforming rockfill embankments have included strict controls for construction and material requirements including all of the following:

- Use of durable, high quality rock. On-site visual classification of material was included. Project specifications were required for slake durability testing of the rockfill embankment material.
- Reduced maximum rock size and decreased lift thickness. The smaller maximum rock size better distributed compaction efforts across the materials, rather than acting on the points of a few large rocks. Projects incorporating a 12” compacted lift demonstrated successful performance.
- Incorporated a strong construction control program specific for rock embankments in the project construction documents. Included requirements for wetting the rockfill embankment material to facilitate compaction.
- Incorporated the use of geotextiles in the construction of the rock embankment. At a minimum, geotextiles are to be used in the rockfill embankment beneath structure foundations to a depth of two times the width of the structure foundation following a 1H:1V projection from the bottom of footing.
- Placement of additional geotextile within the rockfill embankment. Aided in preventing migration of smaller material within the rockfill embankment, had a reinforcing effect, and aided in observing a consistent maximum compacted lift thickness.
- A structural designer included a mechanism for jacking and levelling the superstructure in the final plans.

If the estimated settlement including both the rockfill settlement under self-weight and any additional settlement of the existing subsurface soils within the identified area of influence is expected to exceed tolerable limits (especially, structural tolerances like angular distortion limits), shallow foundations are not allowed, regardless of the confidence in the laboratory testing program’s strength, geotechnical parameters selected, and calculated settlement.

7.13.2.2 Monitoring for High Rockfill Embankments

For either of the two design approaches previously discussed, the following construction protocols are strongly recommended when founding structures on rockfill embankments:

- Implementation of a settlement monitoring program
- Generous float in the project schedule in the event of ongoing settlement or the need to monitor settlement following the quarantine.
- Inspector on-site to witness placement of material and not just during acceptance.
- Continued settlement monitoring beyond quarantine period in the event of post quarantine settlement.
- When possible, test finer material zones within the rockfill, (e.g., pile window backfill, geosynthetically reinforced zone aggregate, etc.) for moisture content and density.

Regardless of the foundation type selected, it is required that full time field inspection is present to confirm that soil and rock is mixed and wetted adequately, the water content and density of the aggregate within the pile window zone and any other granular portions of the embankment are tested, the material meets type, gradation, and durability requirements, and the rockfill is properly placed and compacted as specified in Publication 408, Section 206. This is critical to the post-construction embankment performance. Any deviations from the standard requirements must be submitted via a Special Provision for review and approval by the District Geotechnical Engineer (DGE).

Furthermore, consider monitoring the settlement of the rockfill both during and after construction. A variety of factors affect the quarantine period durations (e.g., changes in hydrostatic conditions, rockfill gradation/durability, construction controls, monitoring, installation procedures, etc.). During construction, the contract special provisions must clearly state:

- The recommended duration of the quarantine period
- The requirements controlling the quarantine period duration
- The frequency of monitoring data collection
- The individual responsible to collect, receive, analyze/interpret the data
- The data format and how to interpret the data
- The individual approving further construction activities

Additionally, project special provisions must clearly state and describe the purpose of the piezometer installation and the requirement to monitor groundwater pore pressures to ensure embankment foundation stability. Special provisions must include minimum qualifications for personnel analyzing and interpreting the piezometric data. It is recommended that the geotechnical designer of record who performed the analysis and prepared the design recommendations conduct these activities.

Discuss the monitoring plan during the preconstruction meeting to finalize any potential coordination issues. At a minimum, the Contractor, Engineer, and Instrumentation Engineer must be present for the discussion. Continuation of monitoring activities post-construction may provide an early indication of unanticipated settlement past the quarantine period; however, also consider the cost/benefit of extending the settlement monitoring period. As discussed in Section 7.10, settlement monitoring devices (e.g., plates, points, piezometers, etc.) can be lost once the foundation is complete and construction of the superstructure begins. Therefore, carefully select the locations of proposed settlement monitoring devices, or alternatively, factor in requirements to re-establish settlement monitoring devices into the project cost and schedule.

Note: A standard Settlement Monitoring Special Provision is currently under development. This will detail the placement, calibration, performance, and use of equipment to measure and monitor settlement and creep of successive lifts of embankment or structural fill. A Special Provision will allow for both manual and automated monitoring methods. Any deviations from the Settlement Monitoring Special Provision will need to be submitted to the DGE for review and approval.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 8 – ROCK CUT SLOPE AND CATCHMENT DESIGN

CHAPTER 8 – TABLE OF CONTENTS

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 (i.e., 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 that 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; therefore, they 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 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 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 Provinces[1]

Geologic Characteristic	**REGION I** **Appalachian Plateaus and Central Lowlands Provinces**	**REGION II** **Ridge and Valley, Piedmont, and New England Provinces**
Bedding or Foliation	Horizontal to Near Horizontal Bedding, typically less than 30º	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 70º-90º 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.

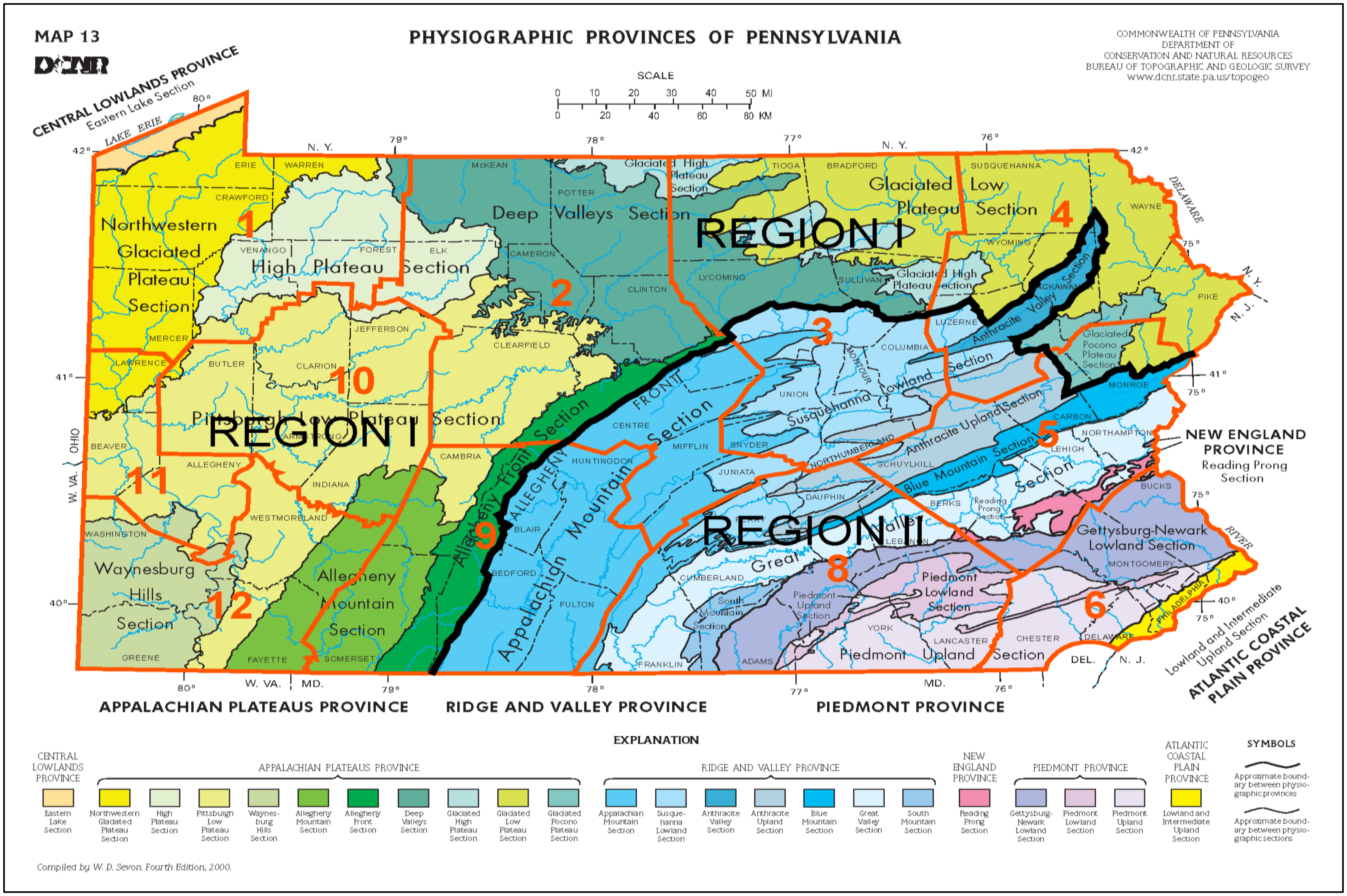


Figure 8.1.3-1 – Physiographic Province Map of Pennsylvania, with PennDOT Districts[1]

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, etc.).

   - **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" according to 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" according to Publication 222, Chapter 3.6.4(f), and having a second cycle (Id_2) Slake Durability Index (SDI) value of 85% or greater according to ASTM D4644.
     - Any siltstone with a second cycle (Id_2) SDI greater than 85% according to ASTM D4644.

   - **Non-Durable Lithologic Unit** – A lithologic unit described as a shale, claystone, or a lithologic unit described as "very soft" to "soft" according to Publication 222, Chapter 3.6.4(f), with a unit weight less than 140 pcf, and with an Id_2 SDI value less than 85%.

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% 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% 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% to 90%. 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^{o}E, etc.).
   - 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).

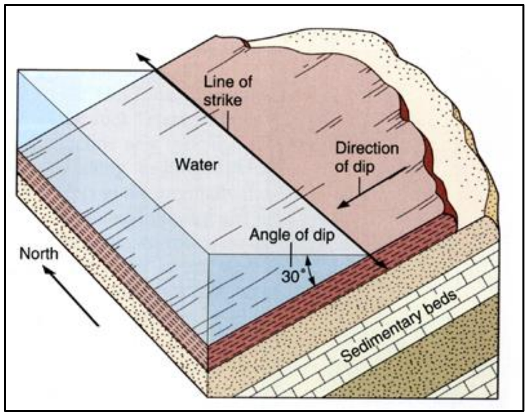


Figure 8.2-4 – Strike, Dip, and Dip Direction Diagram

- 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 (FS).

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 (e.g., 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 it must not "daylight" in the slope face, and the dip of the discontinuity is less than the angle of friction along the discontinuity.
- Cut slope geometry is determined to be according to 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).

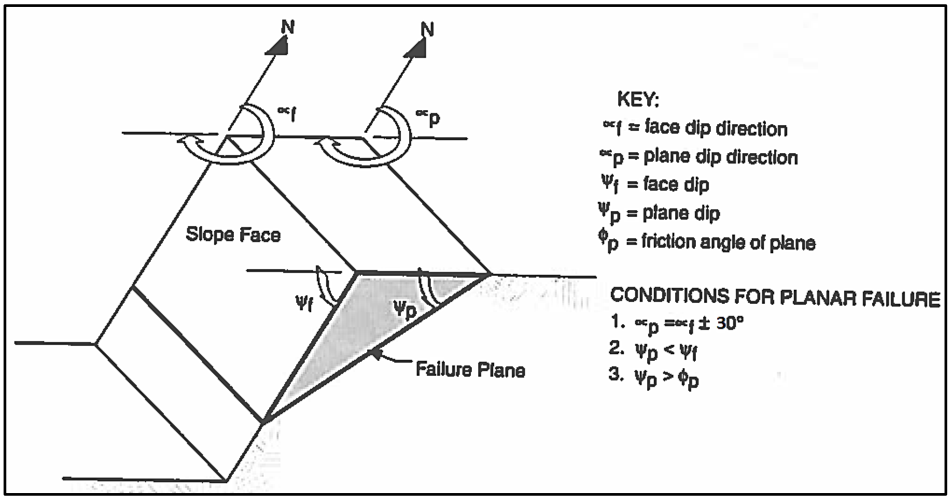


Figure 8.2-5 – Diagram Detailing Conditions for Planar Failure

- 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 (e.g., 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 according to 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 (i.e., 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 to decrease the risk to Department facilities and public safety. Like 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 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 according to 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, etc.)
- 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 according to 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, etc.)?
- 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. 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 ft^2 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 instead 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 according to Chapter 3 of this publication and Publication 222. The subsurface investigation may include angled boreholes and downhole video/acoustic televiewing 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. 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 that can be used for engineering design.

Ground-based LiDAR scanners can also be used for outcrop mapping. 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 that 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 FS 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 FS 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	• 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	• 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	• Use Rock Quality Designation (RQD) and slake durability data • Perform kinematic analyses as indicated by field conditions	• 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.

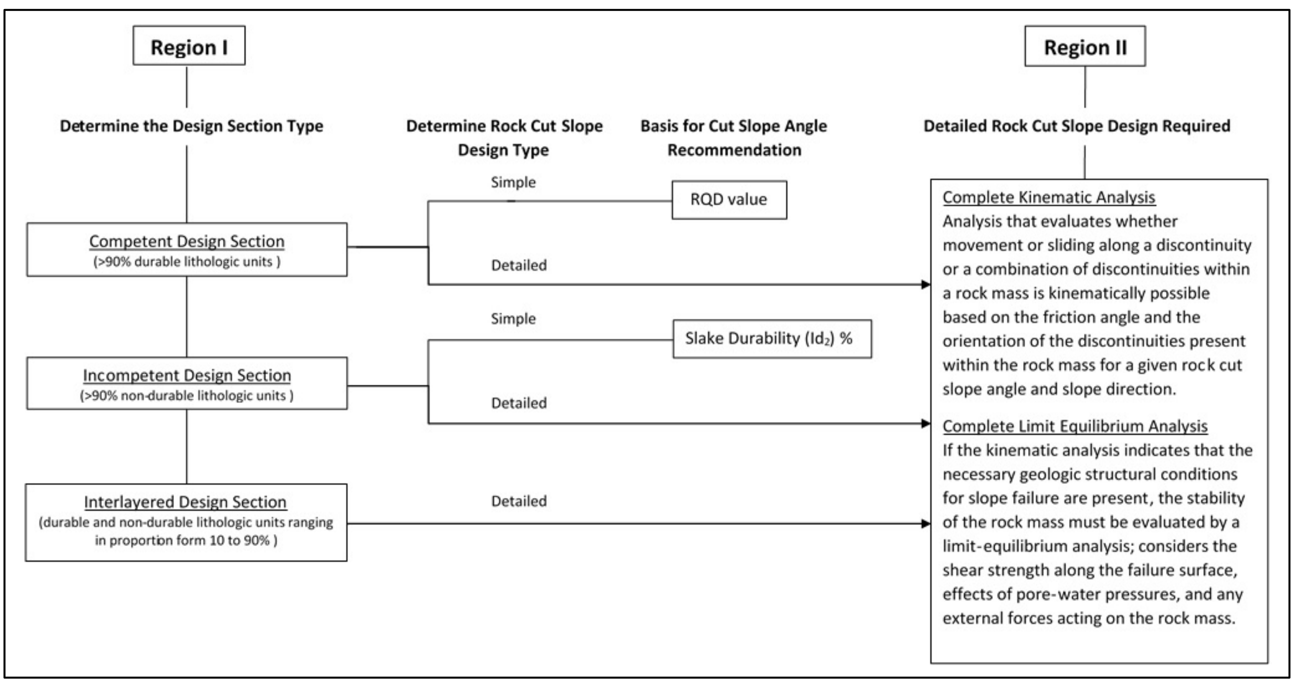


Figure 8.5.1-1 – Rock Slope Design Summary Flow Chart[1]

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 enough 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 that 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) according to Table 8.5.1.1.1-1.

Table 8.5.1.1.1-1 – Cut Slope Angle Recommendations for Competent Design Sections[1]

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 that 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 D4644 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 Sections[1]

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 that 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%. 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 a 3:1 or 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, 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 (Id_2 >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 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 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), etc.) 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 and the Transportation Research Board’s Report on Rockfall Characterization and Control.

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 FS 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 cyclothemic units, which consist of a vertical repetition of sandstone, siltstone, shale, limestone, claystone, and coal. The cyclothemic 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 enough 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%, with a minimum grade of 3%. 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 near each other (i.e., 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. Enough 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, 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. 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 construct the slope adequately or economically, 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 Richie 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.

As mentioned, the ditch substrate material is an important design consideration. For a ditch profile cut into hard bedrock, lining with an energy-absorbing substrate layer (e.g., 12 inches of compacted soil, Class R3 rock, etc.) can enhance the modeled rock containment of the the ditch, or may allow a reduction in ditch dimension and subsequent required right-of-way width. When selecting and modeling a ditch substrate material, the designer should carefully consider the advantages and disadvantages, such as future maintenace cost and required effort to maintain the as-designed lining if the ditch was to be cleaned of rockfall debris.

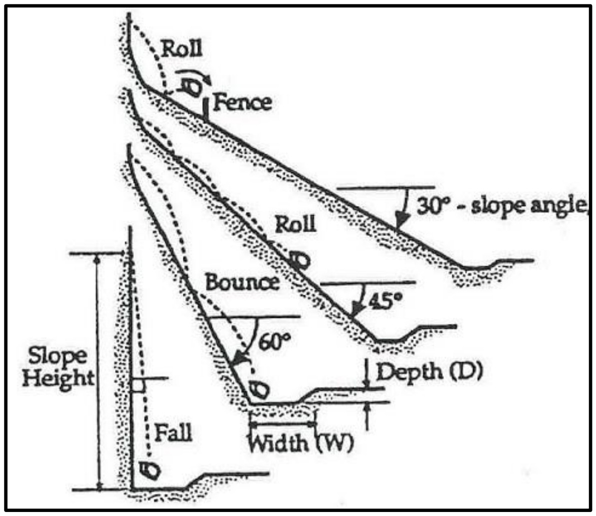


Figure 8.7-1 –Rockfall Trajectory based on Slope Angle[1]

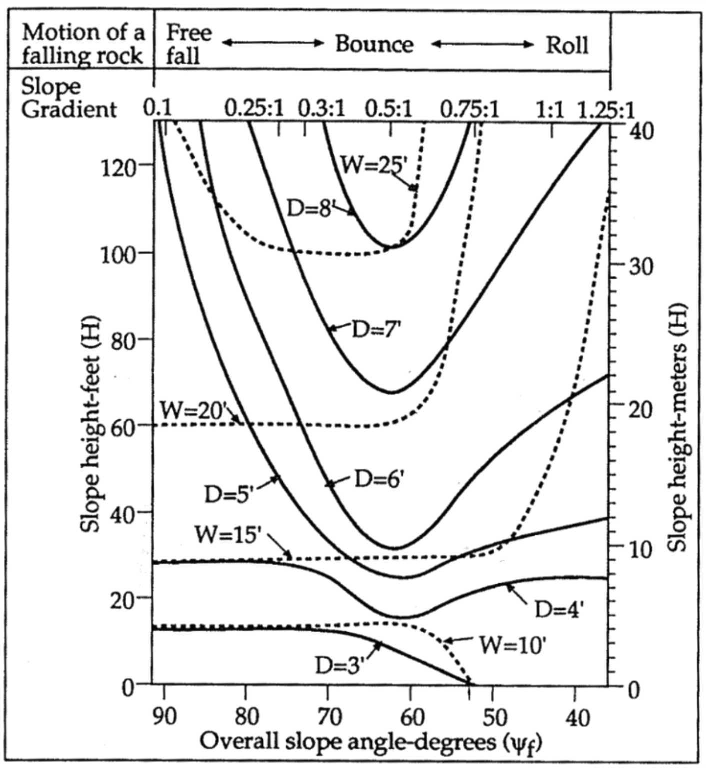


Figure 8.7-2 – Ritchie's Rockfall Catchment Ditch Design Chart[1]

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 (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 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 that are normally required to complete a rockfall simulation model:

- 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. It is recommended to model spherical shaped rocks 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. It is recommended to 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:

$$\theta_{max} = tan^{-1}\left(\frac{S}{R}\right)$$

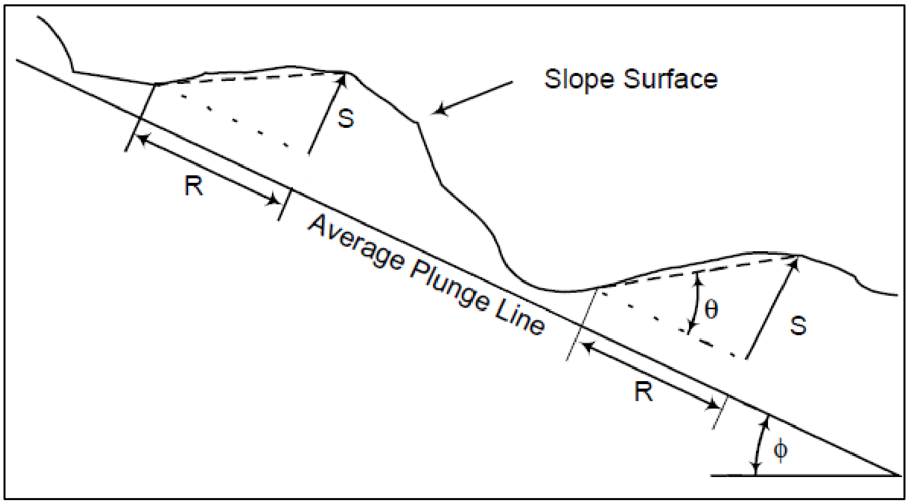


Figure 8.8-1 – Surface Roughness Diagram

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, 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 near 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.

- When an energy-absorbing substrate is considered to line the bottom of the ditch, necessary adjustments to the normal and tangential coefficient values selected for the CRSP model may be required.

Table 8.8-1 – Hardness Reference Guide with CRSP Coefficient Values[1]

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 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-deep by firm pressure of a knife or pick point. Can be excavated in small chips to pieces about 1-inch maximum size by hard blows of the point of a geologist's pick.	0.20	0.80
10	Moderately Strong	Can be scratched with a knife or pick. Grooves or gouges to ¼ inch deep can be excavated by hand blows of a geologist's pick.	0.25	0.85

Hardness Input Code	Consistency/ Hardness	Field Identification	Normal Coefficient Values (R_n)	Tangential Coefficient Values (R_t)
		Requires moderate hammer blows to detach hand specimen.		
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 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 limitations 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 can include 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. When planning the volume of rock to be removed, the project designer needs to consider potential constructability and safety concerns. The extent of planned removal must be enough to accommodate the anticipated drilling and excavation equipment. For any slope area needing rock removal, the Designer should also assess any overlying rock outcrops and adjacent areas of questionable stability that may impact worker safety during construction. Construction plans and specifications that identify and specify the extent of rock removal with a reasonable degree of accuracy is key to a successful project.

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. Hand scaling methods are advantageous because they can target and remove only the rocks identified, causing minimal to no disturbance of the remaining slope material. 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.

The following scaling methods can remove rocks and produce the desired effect of rock slope scaling, but tend to leave a damaged slope behind; therefore, they should not be used:

- Hydro-scaling (using high-pressure water spray)
- Drag Scaling (dragging heavy metal objects across slope)
- Impact Scaling (hitting slope with wrecking ball)

During construction, it is recommended an experienced geotechnical inspector or specialist be onsite full time and not just at the end of the project when it may be too late to correct deficiencies. The inspector must be qualified to discern proper scaling techniques and safe practices, know when the scaling effort has adequately reduced the rockfall risk, and ensure the slope is not over-scaled.

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 that 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 used 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 used 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 that 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 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 that provide both well-defined intent (specifications) and illustrations (plans and details). For example, with moderate and large-size scaling projects, the planned quantity of rock to be trimmed and scaled should be determined by using traditional survey data, aerial photography, or LiDAR Imagery to develop accurate cross-sections. The special provision should also include when the surveying should be performed (i.e., when trees and brush are free from foliage and before snow covers rock outcrops, formations, and other geologic features of the area to be scaled). Plans should show removal limits and include marked-up photographs or 3-D images.

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 above, commonly used geotechnical details are:

- Shotcrete Facing Details
- Anchored Mesh Bolt Details
- Anchored Wire Mesh Facing Details
- Horizontal Drain Details
- Rockfall Fence Details
- Slope Drapery Details

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 9 – COMPACTION OF EMBANKMENT AND FILLS

CHAPTER 9 – TABLE OF CONTENTS

9.1 INTRODUCTION

This chapter of the publication provides background, theory, discussion, and best practices for understanding compaction and the role it plays in the construction of highway and transportation facilities. Compaction of soils and aggregates is one of the most common operations in highway and bridge construction. Practically every project is impacted by the placement and compaction of embankments and fills. Whether the project is a simple bridge replacement or a roadway on a newly developed alignment, compaction of soils and aggregates will need to be performed. It is important that both design and construction personnel understand the theory and mechanics of compaction, as well as the physical and environmental factors that affect it, because of the frequency of compaction operations in construction. A lack of understanding these compaction principals can lead to severe issues during construction; including poor quality of construction that may affect the long-term performance of the project.

9.1.1 Purpose

The purpose of compaction is to improve the engineering properties of the soil mass. By definition, soil compaction is the process of displacing air from the pores between soil grains by applying stress to cause densification. Compaction improves the engineering properties of soil. These improvements include the following:

- Increased shear strength
- Reduced compressibility
- Decreased permeability (for subgrade)

The improvement of these engineering properties results in the following benefits:

- Reduced settlement
- Improved slope stability
- Improved bearing capacity
- Reduced volume changes
- Improved frost resistance

The benefits of compaction do not affect all soils to the same degree. In certain cases, the Engineer must carefully consider the effect of compaction on the engineering properties of project soils. The three key engineering properties (i.e., shear strength, compressibility, and permeability) that are improved by compaction are discussed in brief in the following subsections below.

9.1.1.1 Shear Strength

One of the primary benefits of achieving good compaction is the increase in shear strength of the material. The shear strength of a material is the ability of the material to resist forces that tend to produce a sliding failure along a plane in the material that is parallel to the applied force. In other words, it is the magnitude of shear stress that the material can sustain without yielding. Two primary benefits of increasing the shear strength of a material include improved slope stability and increased bearing capacity.

The shear strength of a soil or aggregate is a function of three parameters: internal friction angle, cohesion, and effective overburden stress. The equation for shear strength is as follows:

$$s = c + \sigma' tan\phi$$

where:

s = shear strength (psf)
c = cohesion (psf)
σ' = effective overburden stress (psf)
ϕ = internal angle of friction (soil or aggregate) (degrees)

The equation can be separated into two components. These two components of shear strength are cohesion and friction. The cohesion component, c, is independent of interparticle friction. Cohesion can be further broken down into two types. True cohesion can be described as the part of the total shear resistance between particles that is independent of the normal force that is pushing the particles together. True cohesion can develop between soil particles that have remained in contact for a long period of time such as stiff overconsolidated clays or by cementing, which turns sand into sandstone. Apparent cohesion can be caused by negative capillary pressure, pore pressure response during undrained loading, and tree roots/vegetation. Generally, the cohesion portion of the shear strength equation is commonly ignored when determining shear strength for design purposes because the magnitude of true cohesion is very small, and the apparent cohesion is unreliable and often lost over time.

For the frictional component, σ' (tanφ), of the equation, the shear strength is dependent on the effective overburden pressure and the internal friction angle of the material. In other words, the frictional component is stress or load dependent. As the effective overburden pressure increases, the shear strength increases as well. Compaction of the material forces the particles closer together, which increases the unit weight of the material and the particle-to-particle stress. This increases the frictional component of the shear strength equation.

9.1.1.2 Compressibility and Permeability

Compressibility is defined as the susceptibility of soil to decrease in volume when subjected to load. Reducing the compressibility of embankments and fills is critical to limiting total and differential settlement to acceptable levels. Compaction reduces air voids between the soil particles resulting in a more dense, uniform material that is less susceptible to total as well as differential settlement.

For transportation related projects, a high permeability, free-draining material is normally desirable from a geotechnical engineering perspective, especially as it relates to pavement subgrades. Unfortunately, subgrade soils in Pennsylvania typically have a high proportion of fine-grained particles (i.e., material passing the No. 200 sieve) or are classified as fine-grained materials. The fine-grained soils commonly encountered throughout the state have lower permeability than coarse-grained materials. Rather than viewing the lower permeability of the typical subgrade soil as a deficiency, designers should use this to their advantage where applicable.

Compaction reduces the permeability of soil, which creates a greater resistance to the flow of water through the material. Compacting a fine-grained soil, which is already poor at draining, will reduce the permeability even further. The reduced permeability of the compacted subgrade becomes advantageous because it provides greater resistance to damage and weakening associated with water infiltration and frost action.

Any water that enters the pavement should be removed through the subbase layer. The subbase has a much higher permeability and can transmit water laterally to the pavement base drain or at the edge of the embankment slope if the subbase layer daylights on the embankment slope. The less permeable subgrade surface underneath the subbase should be graded to allow for drainage toward the outside edge of the pavement.

9.2 MECHANICS OF COMPACTION

Compaction is the densification of soils and aggregates by the application of mechanical energy. Compaction occurs immediately upon the application of the load, and density of the material is measured as unit weight. As mechanical energy is applied to the material, air is displaced from the pores between the soil grains. The reduction in air voids results in densification of the material and an increase in the unit weight. Simply put, compaction is the reduction of air voids by the application of mechanical energy. This is shown graphically in Figure 9.2-1.

It is important to understand that the amount of water present in the material has a significant effect on the ability to achieve compaction. Water acts as a lubricant between the soil particles so that they can move during compaction. This helps to improve particle rearrangement. The greatest density or level of compaction that can be achieved occurs at the optimum moisture content of the material. However, if the amount of water in the material is too great, compaction will be difficult to achieve because water is filling the void spaces that should have been filled by soil particles. If the amount of water in the soil is too little, re-arrangement between the particles will be inefficient. Therefore, the control of the moisture content of the material is critical to achieving adequate compaction.

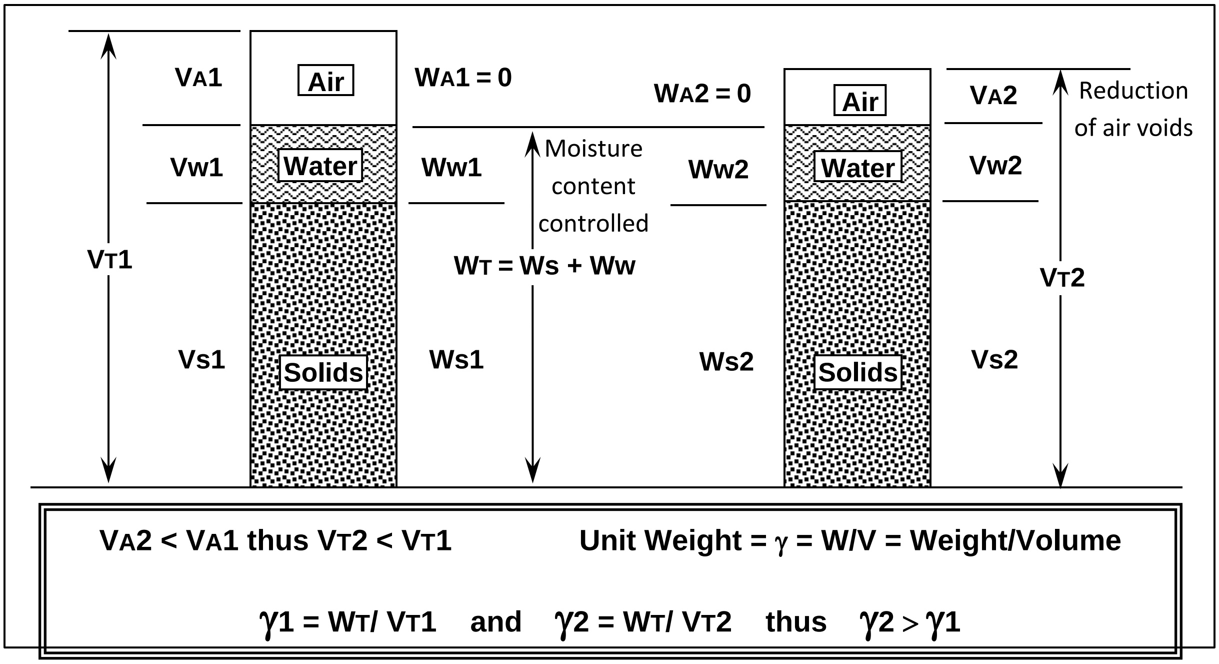


Figure 9.2-1 – Unit Weight (Density)

9.2.1 Types of Compaction

Regardless of the method of compaction, efficient densification of soils and aggregate (i.e., reduction of air voids) is achieved by shearing of the material particles. For a volume change to be achieved efficiently, it is not enough to just simply apply a load to compress the material. Instead shear between the soil or aggregate particles must occur locally. The term "locally" is stressed because it not the intent to cause a general bearing capacity failure, but rather locally move the particles relative to one another into a tighter, more compact, arrangement. This can best be visualized by considering the difference between a generalized bearing capacity shear failure and punching shear (refer to Figures 9.2.1-1 and 9.2.1-2).

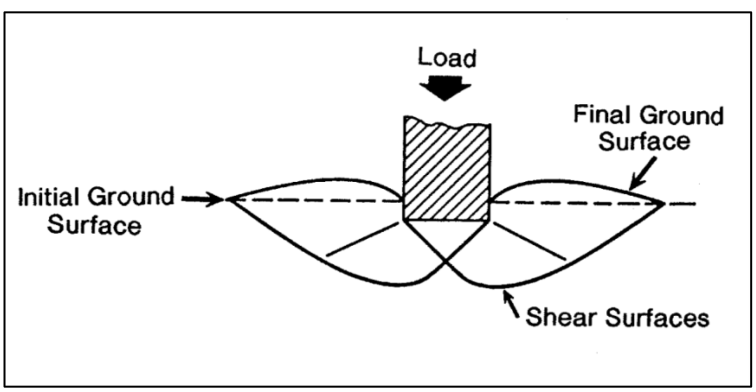


Figure 9.2.1-1 – Generalized Bearing Capacity Shear Failure

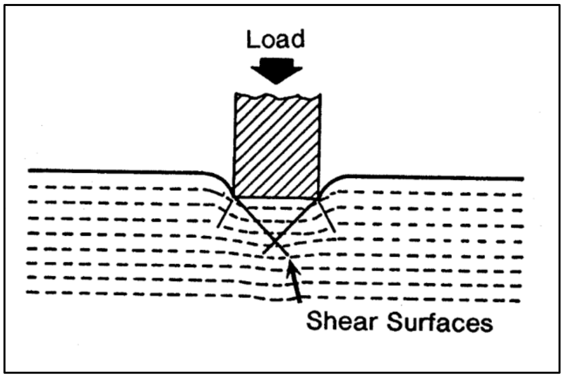


Figure 9.2.1-2 – Punching Shear

As shown in Figure 9.2.1-1 (Generalized Shear), as the ground fails, shear surfaces develop resulting in heaving of the ground surface on either side of the loaded area and no volume change. In Figure 9.2.1-2 (Punching Shear), the load punches into the bearing material, and there is no ground surface heave. The load causes the soil particles to locally shear relative to one another resulting in a volume change. The volume change signifies densification or compaction of the material.

9.2.1.1 Static Compaction

Static compaction is simply the force applied by the dead weight of the equipment, which acts to expel air voids allowing compression of the soil particles. The effective compaction energy is controlled by adding or subtracting weight (ballast) to or from the equipment, or if possible, changing the contact area to change the contact pressure. Discretely applied loads are most effective and efficient for static compaction. This is most efficiently achieved by use of a padfoot type roller. Static compaction is appropriate and effective for fine-grained materials, including materials with as little as 20% fines (i.e., 20% passing the No. 200 sieve).

9.2.1.2 Vibratory (Dynamic) Compaction

Vibratory compaction (also referred to as dynamic compaction) uses an engine driven mechanism to create a dynamic force in addition to the static weight of the equipment. The dynamic load mechanism is usually a rotating eccentric weight inside the drum (for rollers), an eccentrically weighted shaft called an exciter (for plate compactors), or a piston/spring assembly (for rammers). Vibratory compaction is appropriate and effective for cohesionless (non- to low plastic) granular soils and aggregates having a low fines content (i.e., less than 20% passing the No. 200 sieve). The cyclic (rapidly repeating) dynamic load results in vibration of the material. The vibrations cause a temporary reduction in shear strength of the granular material, enabling the combined static (equipment weight) and dynamic forces to rearrange the granular particles into a denser arrangement.

9.2.2 Compaction Based on Material Type

As stated above, localized shear is necessary for effective and efficient compaction operations. The desired mechanism for locally shearing the particles varies for different material types. Methods and procedures effective for one type of material may be highly inefficient or potentially damaging for another soil type; therefore, it is important to use appropriate compaction equipment based on the material type. The following subsections discuss compaction based on general material types.

9.2.2.1 Compacting Coarse-Grained Cohesionless Materials

Coarse grained materials with low fines content, (i.e., less than 20% passing No. 200 sieve) have generally good permeability; therefore, pore pressures between the soil particles are easily dissipated, and water is not a problem during compaction if it is maintained near the optimum moisture content. During dynamic loading of granular materials, the particles vibrate resulting in significant loss of particle to particle stress (i.e., the pressure between particles reduces). In other words, temporarily, while the material is vibrated, φ is greatly reduced. Since $s = \sigma'(\tan\varphi)$, when φ is reduced, there is a temporary loss of shear strength. During this temporary loss of shear strength, the combined static and dynamic loads locally shear the soil particles into a denser arrangement. As the roller moves forward, the vibrations cease, and the material locks into the denser configuration. The particle to particle contact and stress is restored, and the material regains its shear strength (somewhat increased from the increased contact stress between particles). This is the process of densification with vibratory compaction for coarse grained cohesionless materials.

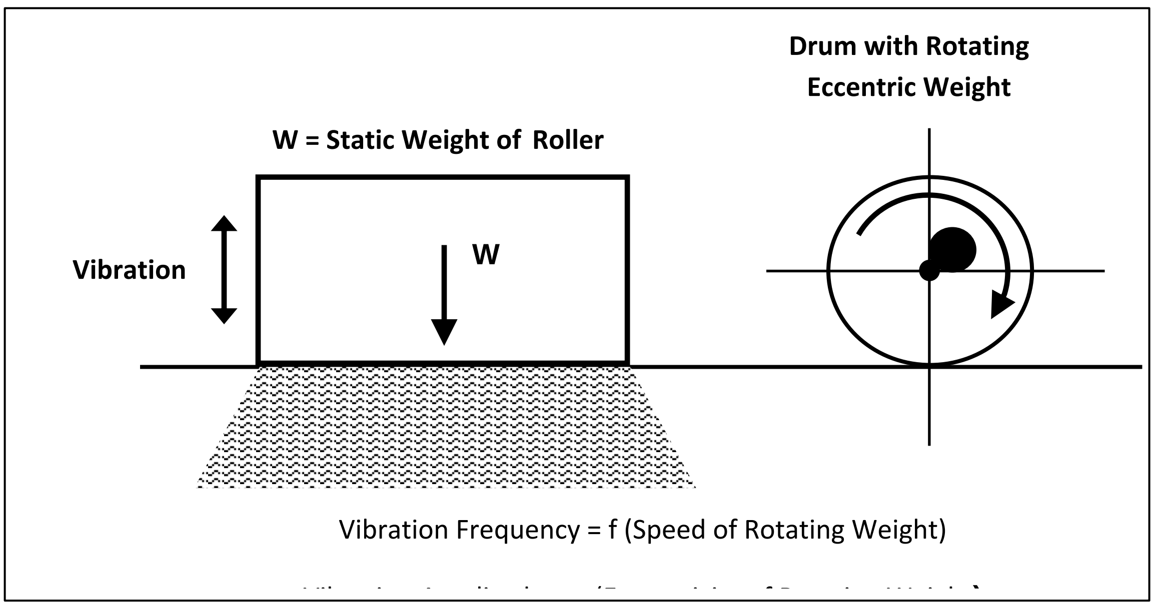


Figure 9.2.2.1-1 - Vibratory Compaction

9.2.2.2 Compacting Fine-Grained Cohesionless and Cohesive Materials

The compaction process is different for granular materials with significant fines content (i.e., materials with 20% or greater passing the No. 200 sieve) and fine-grained soils. Fine-grained soils have much lower permeability. At optimum moisture and maximum dry density, soils are approaching saturation with percent saturation values approximately 95%. At these high percent saturation values, rapid load conditions (such as with compaction equipment) generate excess pore pressures. Since water is not compressible, and fine-grained soils have low permeability, when load is applied, the water pushes back against the soil since the water cannot rapidly drain out of the soil.

Vibratory compaction only further increases pore pressures, especially in cohesive materials, which are energy sensitive. Even in static mode, the use of a smooth wide-drum roller on fine-grained soils often result in the equivalent of a bearing capacity failure (pushing and shoving) with heaving in front of and behind the drum (i.e., also considered as pumping referenced in Section 9.5.3). To avoid this problem, fine grained soils need to be compacted with discrete static loads. The loading is considered static in the sense that there is no vibration. To accomplish discrete static loads, use a padfoot type roller.

Recall that compaction is the reduction of air voids. At optimum moisture and maximum dry density, soils are approaching saturation. Since air and water move through the same network of interconnected pores and pathways, and fine-grained soils have low permeability (i.e., water does not move freely through a fine-grained soil mass), the movement of air through soil becomes increasingly restricted as soil is compacted. By applying compaction loads discretely rather than a line load like a smooth drum roller, the unloaded areas between the feet or pads of the roller serve somewhat like a pressure relieve valve. The internal pore pressure from incompressible water and compressed air generated in the loaded area can bleed off into the unloaded zones allowing air to escape and compaction to occur. The discrete loading shears soil particles locally much like punching shear that results in a greater permanent deformation and volume change. In contrast, general shear in the bearing capacity of spread footings is primarily elastic and produces little volume change

9.3 COMPACTION EQUIPMENT

Equipment used for compacting various earth materials can range from small, hand-operated equipment to large rollers. Depending on the type of material being compacted, the compaction equipment plays a significant role in the compaction process. Equipment requirements for compaction equipment can be found in Publication 408, Section 108.05. Permissible compactor types based on the material being compacted are specified in Publication 408, Section 206. The following subsections briefly discuss the general types of compactors.

9.3.1 Dynamic Compactors (Vibratory Roller and Hand Operated Equipment)

Dynamic compactors are well suited for the compaction of granular (cohesionless) materials. Vibrations temporarily reduce interparticle shear strength allowing particles to arrange into a denser state by void reduction. The compactor efficiency depends on:

- Ballast (weight)
- Roller Speed
- Vibration Amplitude
- Vibration Frequency
- Drum width

Compaction is primarily from the dynamic force created by the compactor rather than static weight. The dynamic force is proportional to square of vibration frequency. There is a large reduction in efficiency without vibration (static mode) when compacting granular materials. Note that the efficiency is achieved by shearing the soil, not simply by compression. Dynamic compactors should not be used when compacting cohesive soils or non-plastic fine-grained soils unless they are only used in the static mode at the end of the workday to seal the surface of the embankment to promote positive drainage.

9.3.2 Static Compactors (Padfoot Rollers)

Padfoot rollers are the best option for compacting cohesive soils (clays) and non-plastic fine-grained soils (silts). The pads provide "kneading" action. The process of "kneading" is shearing the soil, which results in volume change. As the compactor repeatedly passes over the material being compacted, the penetration of the pads into the soil decreases as density of the soil increases with each pass, "walking up", out of the soil. The number of passes required for the pads to "walk out" (where the entire weight of the roller is supported on the pads resting on top of the fill) establishes compaction requirements. If pads do not "walk out" or the pads are picking up soil, the equipment is too heavy, the material is too wet, or the pads are not creating a change in volume by shearing the material. In these cases, modifications to the material, such as scarifying and drying, are typically needed to achieve the required compaction. A benefit of padfoot rollers is that they compact from the bottom of the lift upwards to the top of the lift and when the pads penetrate through the lift, they compact the lift below.

9.4 LABORATORY AND FIELD TESTING FOR COMPACTION

9.4.1 Laboratory Moisture-Density (Proctor) Testing

The moisture-density (M-D) or Proctor test method was developed by R. R. Proctor in 1933. Publication 19, PTM 106, which is a slight modification to AASHTO T99 (Standard Compaction), is typically used to simulate field compaction for embankment, subgrade, and foundation construction. 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. Moisture-density testing provides a target maximum dry density achieved at the optimum moisture content for the compactive effort applied. Moisture-density is a quality control tool to aid in obtaining proper compaction of soil used for construction. The relationship is only valid for finer grained soils (silts and clays that may contain some sand and little gravel or sands with silt and clay). The test is also valid for clean sands. It is not considered appropriate or recommended for gravels and coarse aggregates because the material is too large compared to the size of the mold. Also, these materials have no cohesion and higher permeability; therefore, water runs through the materials and out of the bottom of the mold resulting in an unreliable curve.

Density obtained is a function of three variables: soil type, compactive effort (energy), and water content. The type and magnitude of the compactive effort is also important. The combination of these three variables will produce a unique moisture-density relationship (i.e., compaction curve). This relationship between the variables is unique; therefore, when the soil type, compactive effort, or water content changes, the moisture-density relationship changes. Refer to Section 9.5.2 for guidance when additional testing is warranted.

The basic procedure for performing the Proctor test is relatively straight forward. Using the moisture-density test apparatus (hammer and mold), compact the sample in the mold at a prepared approximate moisture content. Prepare additional molds using increasing moisture contents. Note that the same compactive effort is to be used to prepare each mold. The results yield a moisture-density relationship typically in a "horseshoe" shaped curve as shown in Figure 9.4.1-1. The optimum moisture content (w_{opt}) and maximum dry density (γ_{dmax}) are then selected from the generated curve. The maximum dry density is the highest point on the curve and the optimum moisture content is the moisture content associated with that point. These values are the basis for target density values that must be obtained during compaction, while maintaining the moisture within a specified band around the optimum moisture content. Refer to PTM 106 for more details regarding the testing procedure.

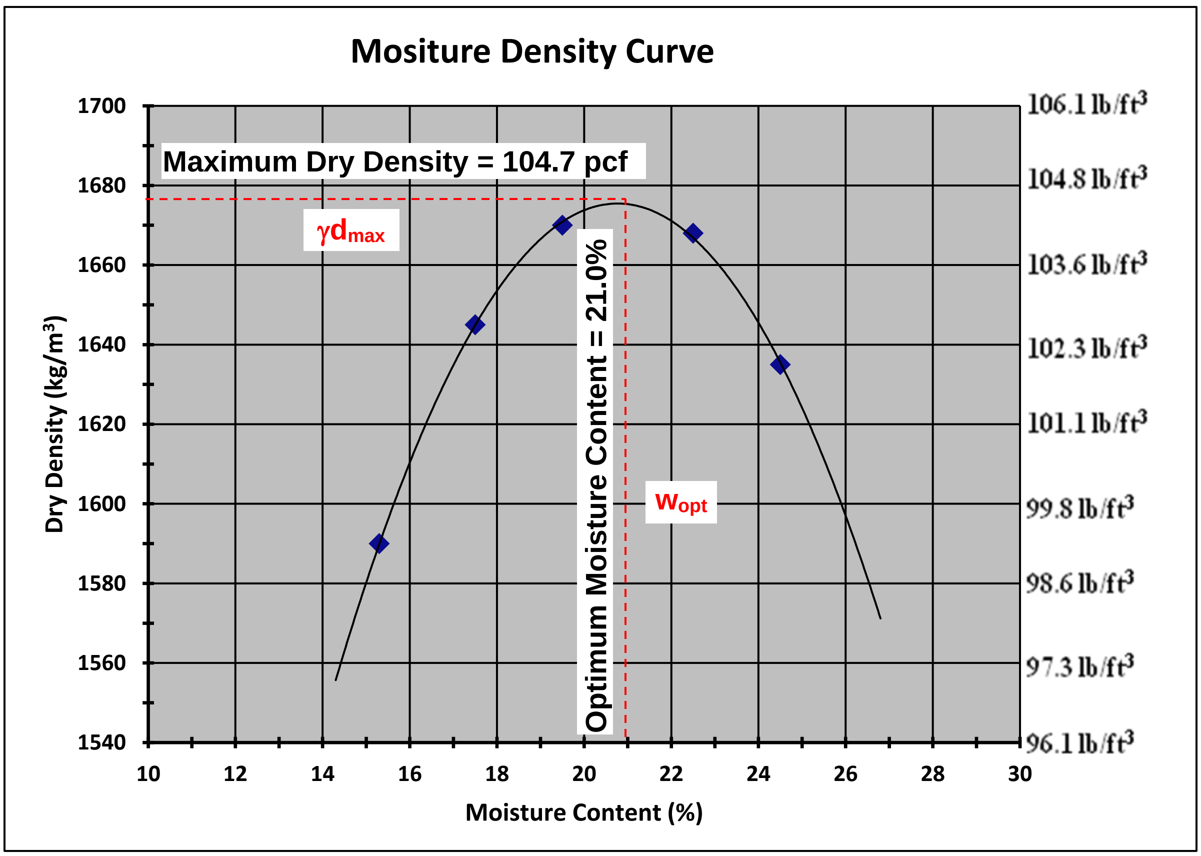


Figure 9.4.1-1 – Typical Moisture-Density Curve

Note that the family of curves associated with Proctor testing is not recommended to obtain a Proctor curve in Pennsylvania due to the inconsistency or variability of the soil across the state and insufficient Proctor testing availability within a geologic region used to establish a family of curves. A family of compaction curves can be developed by plotting numerous Proctor curves from a project's region consisting of similar geologic soils of the same origin. These Proctor curves can then be plotted on the same M-D graph. After the family of curves is developed, the results of a one-point test made from the site-specific soil in that region can be plotted on the family of curves. If the point falls on a curve, that curve is selected to represent the soil for compaction and testing in the field. If the point falls between two curves, a new Proctor curve will be drawn to pass through the plotted point and the best-fit curved line relative to the nearest existing curve.

Also, note that the maximum dry density is dependent upon the specific compactive effort (energy) and method of compaction. The value obtained in the lab M-D test does not necessarily reflect the maximum dry density that can be achieved in the field for the same soil with the various compaction equipment that may be used. Light weight, hand-operated or walk-behind compactors may not produce an energy that is equal to the energy of the hammer used during the laboratory test, resulting in difficulty in achieving the maximum dry density from the laboratory test. Conversely, large compactors with energies greater than those produced in the laboratory test may achieve density greater than the maximum dry density determined in the laboratory.

When performing compaction testing in the laboratory, it is important to test the actual materials that are going to be placed and compacted in the field. Do not combine different soil types from a project site to create a "representative" composite sample. Testing representative sample(s) of site soil or proposed fill is not appropriate because the composite sample will not represent any of the individual site materials. Each distinct material that will be placed and compacted in the field should have its own Proctor test so that the density and moisture content can be controlled appropriately.

9.4.2 Field Testing for Compaction Operations

Field testing of compaction operations is normally performed using a nuclear moisture-density gauge. The gauge contains a radioactive source and detector tubes, which detect gamma radiation emitted from the radioactive source and passed through the material being tested. The radioactive source is in the bottom of a source rod. The source rod is located inside the shielded base of the gauge until the operator activates the release mechanism on the handle. Once the release mechanism is activated, the source rod will give off radiation so that the density of the material can be measured. It should be noted that radiation is always being emitted from the gauge; however, when the source rod is housed inside the base with the shielding in place, the amount of radiation given off is very low. Another radioactive source is embedded in the base of the gauge. This source is used to measure moisture within the material being tested while the density is being determined.

The gauge may be operated in two different modes (backscatter or direct transmission) to determine density. The backscatter method is generally used only on bituminous paving, while the direct transmission method is most appropriate for soils and aggregates. In the backscatter mode, the source rod is positioned just above the surface being tested. The depth of penetration of the radiation emitted before being reflected back to the detector tubes is limited to the top few inches of the material being tested; therefore, the backscatter mode is limited to testing thin lift thicknesses and is not appropriate for typical lift thicknesses of soil and aggregates. Refer to Figure 9.4.2-1. The direct transmission method requires forming a hole so that the source rod can be inserted to a specified depth into the material being tested. This method allows for the testing to be performed on materials with greater lift thicknesses (i.e., soils and aggregates). Refer to Figure 9.4.2-2.

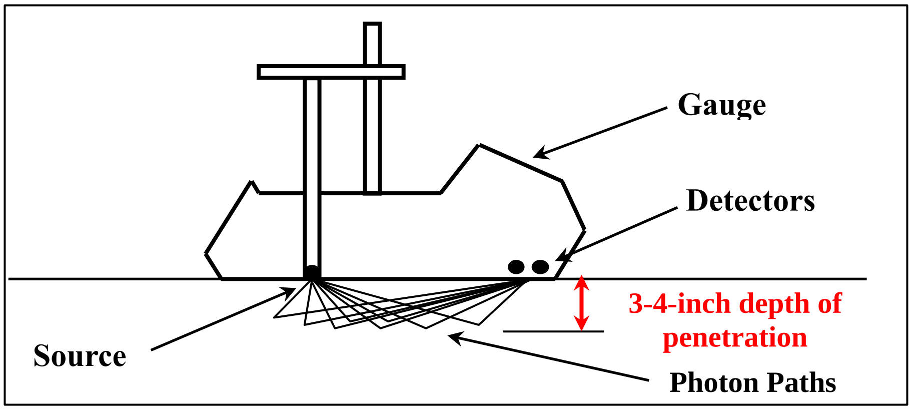


Figure 9.4.2-1 – Backscatter Method

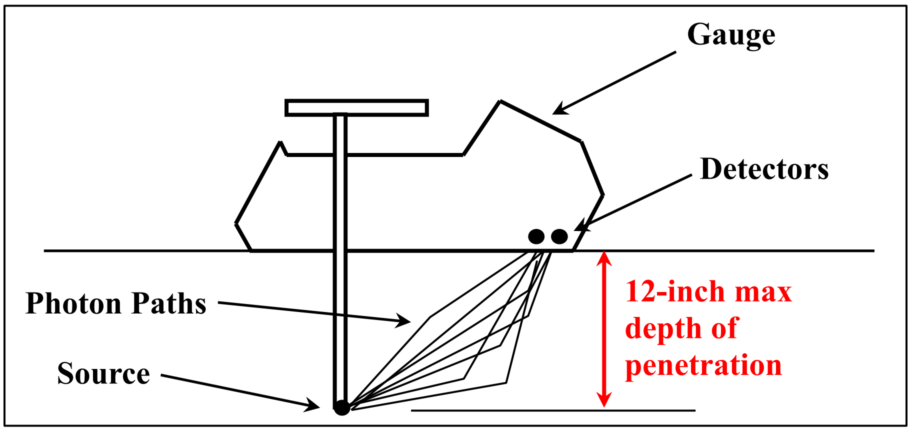


Figure 9.4.2-2 – Direct Transmission Method

When testing soil or aggregate, the gauge is measuring the amount of radiation detected over a set time frame. The lower the amount of measured radiation, the higher the density of the material being measured. The counts recorded by the gauge are converted into a wet density reading. For moisture content, the gauge is measuring hydrogen ions. The greater the count, the higher the moisture content. The in-place wet density and moisture content measured by the gauge are converted within the gauge to dry density for use in comparing the field readings with the Proctor test results.

9.4.3 Laboratory Testing vs. Field Testing

There are several distinct differences between laboratory testing and field testing for compaction. The following subsections discuss these differences in detail.

9.4.3.1 Dry Density vs. Wet Density

The results of the Proctor testing performed in the laboratory are reported as maximum dry density and optimum moisture content. Conversely, the density measured by the nuclear moisture-density gauge is wet density and the moisture content is the in-situ value. Note that dry density and wet density are not the same. In order to compare the field measured values to the laboratory values, the field measured, wet density must be converted to laboratory tested, dry density. This calculation is performed internally by the nuclear gauge and the values for wet density, dry density, and moisture content are reported by the gauge.

9.4.3.2 Nature of Compactive Energy

The nature of the compactive energy used during laboratory testing is highly dynamic, meaning that the compaction is accomplished by dropping a weight from a set height and letting it free fall onto the specimen. This creates a distinct dynamic load on the sample. In the field, the loading of the soil being compacted may range from static loading for fine-grained soils to vibratory loading for granular materials. These differences in the nature of the loading can cause the soil to behave differently. This must be considered when comparing laboratory and field operations.

9.4.3.3 Boundary Conditions

The boundary conditions that define the compaction operations in the laboratory and in the field, are different. In the laboratory during the Proctor test, the sample is placed in a steel mold and compacted. The steel mold is rigid and provides complete confinement of the material as it is being compacted. The level of confinement in the field is not the same as in the laboratory. The confinement in the field is provided by lateral earth pressure applied by the surrounding soil and is much more variable than the consistent and controlled laboratory condition. These differences in the level of confinement can cause inconsistency between laboratory and field densities.

9.4.3.4 Operational Conditions

The operational conditions vary considerably when comparing laboratory and field compaction. The laboratory environment is controlled, moisture is consistent, and material variability is not an issue. In the field, there are no environmental controls, moisture is normally within a band that could vary up to 3% to 4%, and material variability is common. The variability of the operational conditions can lead to variations in compaction performance. Therefore, it is important to maintain good control of the materials being placed regarding material type and moisture content.

9.5 FIELD OPERATIONS

Field conditions that are necessary to achieve a good roadway foundation (subgrade) are:

- Uniform Thickness

- Uniform Material Properties
- Uniform and Dense Compaction
- Proper Compaction Equipment and Technique

Nonuniform conditions during compaction result in variable bearing conditions and differential settlements during service loading. Equally important is good drainage. The greatest care in preparing, placing and compacting the subgrade is of little to no value if it is associated with poor drainage.

9.5.1 Importance of Lift Thickness

When a load is placed on soil, the pressure created dissipates (is distributed) rapidly with depth. For this reason, lift thickness is controlled during placement and compaction. If a lift is too thick, then the stress required to compact the soil will be insufficient at the lower portion of the lift. Lift thickness must be maintained uniformly at or below the maximum specified thickness so that the compaction equipment imparts enough energy to reduce the air voids in the bottom of the lift. Inappropriately thick lifts result in insufficient compaction of material at the bottom of the lift. Refer to Figure 9.5.1-1 to compare lift thicknesses vs operational efficiency.

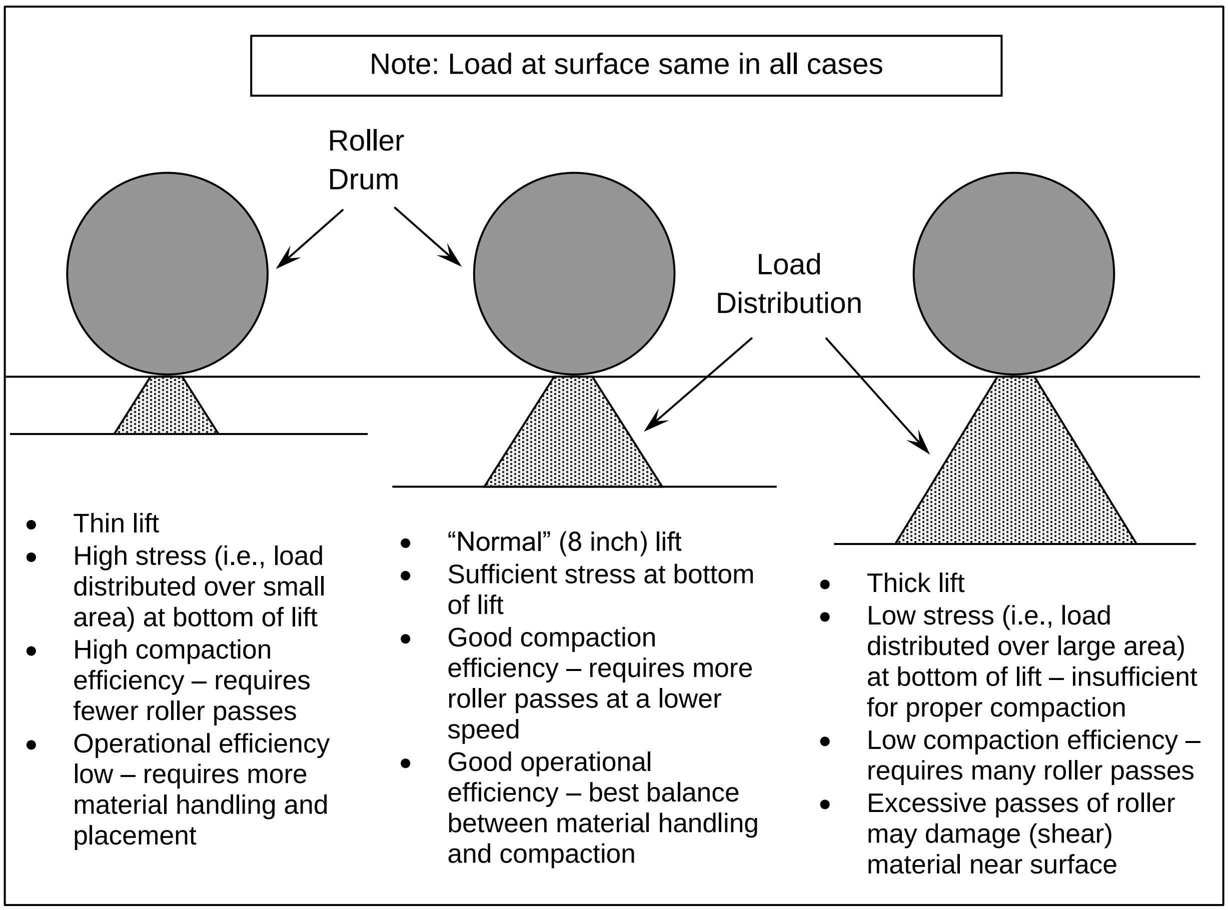


Figure 9.5.1-1 – Lift Thickness vs. Efficiency

Lift thicknesses are typically 4 inches of compacted thickness when using hand operated or walk behind compactors. Thicker lifts are allowed with specified equipment; 6 inches of compacted thickness for materials such as soil, finer graded granular material, shale, random material, and 8 inches of compacted thickness for coarse graded granular material. Lift thicknesses for various materials are specified in Publication 408, Section 206.3(b).

9.5.1.1 Considerations for High Embankments with Rock Fill

Special consideration should be given to the type and quality of material, placement, and compaction of rock fill used in high embankments. Excavated rock used as embankment fill varies across the state with a wide range of durability and hardness. The use of degradable or non-durable material in embankments can result in significant long-term settlement and stability problems as the rock degrades. For more information regarding rock slake durability, reference Chapter 15 of this publication.

Rock fill material in high embankment construction should be placed and compacted as specified in Publication 408, Section 206.3(b). However, consideration given to lift height, type of roller and when it is used, the use of geotextiles, and timeframe of the schedule can contribute to a properly constructed rock fill embankment. When compacting the material, achieving a more effective distribution of the compactive effort can depend on the type of roller used. For example, several initial passes with the use of a padsfoot roller over uneven or large rock followed finally by a pass with a smooth drum roller can increase compaction efforts. Also, to achieve proper compaction and remove or minimize voids or bridging in high embankment construction, focus should be on the placement and inspection of each lift so that rock voids are filled with rock fines and soil. Dumping the fill material on top of the previous layer and working the material into place by blading or dozing can reduce segregation of the larger rocks. If the objective is to increase compaction efforts by reducing the migration of material between each lift, placing geotextile as a separation layer for rock fills may be recommended. However, consider mixing soil and rock to fill voids before and after placement of the geotextile to avoid or eliminate damage to the geotextile when blading and dozing the material. Regarding project schedule, building extra time into the schedule can be advantageous to allow for proper settlement. In instances where time is limited for settlement of the embankment, reducing the specified maximum rock lift and/or conditioning rockfill by wetting should be considered. Settlement concerns and considerations for compacted fills is discussed more in Chapter 7 of this publication.

9.5.1.2 Compaction for Bridge Embankments

Numerous state agencies and FHWA have conducted investigations into structure backfill at bridge abutments and consider embankment settlement and poor compaction of the embankment fill to be the two leading factors. Emphasis of poor compaction during earthwork operations around bridge structures is discussed more below. Backfill material gradation can also impact the integrity of the material compaction. Typical structure backfill materials allowed for use on Department projects may include AASHTO No. 1, AASHTO No. 3, AASHTO No. 5 and AASHTO No. 57 gradations. Note that the large particle size of AASHTO No. 1 and AASHTO No. 3 aggregates, 4” and 2.5” top size, respectively, makes achieving compaction with typical equipment more difficult. Additionally, the following aggregate characteristics, including particle size (i.e., coarse), gradation (i.e., poorly graded), and open texture (i.e., extremely low sand and fines content), tend to have an increased susceptibility to vibrations under loading, which also may result in post construction settlement. Additional discussion on the settlement of compacted fills can be found in Chapter 7 of this publication.

Before construction, designers should take the necessary measures to mitigate potential problems encountered in earthwork construction relative to excavation and configuration of the structures around which the backfill is to be placed. During construction, poor compaction can stem from improper backfilling procedures and/or inadequate compaction control. The following abutment backfill practices, which minimize settlement and increase the effectiveness of compactive effort, are to be employed:

- Use specified/appropriate structure backfill
- Maintain a proper lift thickness
- Use geosynthetics for additional strength, to designate lift height, and to prevent migration of fines
- Verify proper placement and compaction of materials through field inspection

Poor compaction control of embankment material can result in a low density, deformable mass that can continue to settle over an extended period affecting construction schedule, and potential performance/longevity of the structure. Improper compaction techniques are often attributed to difficulty achieving compaction in confined/restricted working spaces, such as behind a bridge abutment. These confined or restricted areas may require the use of smaller compaction equipment, which may result in areas of reduced compaction and density of backfill material. FHWA Report 0-6022-1 provides additional information and recommendations relative to compaction of material for bridge approaches.

9.5.2 Maximum Dry Density Greater than 100 Percent

The optimum moisture and maximum dry density indicated by a moisture-density curve is for a specific compactive effort or compactive energy. As compactive energy increases, the maximum dry density increases and the optimum moisture content decreases. If the compactive energy applied during construction is greater than used during laboratory moisture-density testing, then the material in the field will obtain a greater dry density at the same moisture content. Remember that moisture is controlled during construction and percent compaction is the target. As a result, the material is being compacted consistently with a higher energy moisture-density curve (for the specific material) and is being compacted wet of optimum.

Dissimilar materials exhibit different moisture-density characteristics (the curves will be different). If the soil tested in the laboratory is not representative of the material being compacted on the project, a maximum dry density greater than 100% can be achieved. To avoid this situation and to prevent false results when comparing field tested values with laboratory tested values, control of the materials being placed in the field would be necessary. Whenever material gradation, color, or plasticity of the material changes, a new moisture-density test should be performed to generate the correct moisture-density curve for the material. The new value should then be entered into the nuclear moisture-density gauge before placement and compaction continues.

9.5.3 Heave, Pumping, and Rutting

Common terms associated with compaction in the field are heaving, pumping, and rutting. These field conditions normally indicate that there are issues with the compaction operation. Heave and pumping occur when the compaction equipment causes elastic deformation of the material being compacted. The material deforms upon loading and returns to its original position before the load was applied. Rutting occurs due to punching shear or combined punching and local or general shear. Like heave and pumping, rutting results from a surface shear or bearing failure of soil when the loads applied exceed the shear strength of the soil. This normally occurs because the moisture content is too high for the applied compactive effort.

There are many factors that contribute to the occurrence of heave and pumping. Heave and pumping usually occur when there is excess moisture in the material during compaction. This is especially true for soils with high fines content (i.e., soil has a low permeability) that cannot drain as the load is applied. In this case, the applied load is carried by both the soil and pore water pressure. Repeated loads ultimately result in surface shear or bearing failure of the soil because the applied loads, including excess pore pressures, exceed the shear strength of soil. This results in a degradation of the strength accomplished from all previous compaction.

If signs of pumping occur, there are several things that can be attempted to mitigate the condition. If vibratory compaction is being used, turn off the vibrator and use the compactor in the static mode. Vibratory compaction should never be used on soils with high fines content. Another technique that can be used is to make an initial pass upgrade. Rolling downgrade may cause soil to flow in “waves” in front of the roller. If these techniques do not work, a lighter roller, a padfoot roller, scarifying and drying of the soil by aeration or blending, or the addition of a drying agent, such as lime or cement, may be warranted.

9.5.4 Non-Movement

In order to determine what is classified as non-movement, it is first necessary to understand the type and nature of the movement. There are two basic types of movement: elastic (temporary) and inelastic (permanent). Between the purely elastic and inelastic zones, there is a transitional zone where there is a combination of elastic and inelastic movement. In this zone, the compression (strain) is primarily elastic, but starts accumulating small amounts of inelastic movement or permanent strain.

Assessing non-movement can be difficult and requires careful analysis of the conditions present. Non-movement must be assessed according to the material type and method of compaction. Fines content of the material will impact how non-movement is assessed. For example, generally, small movements (under compaction or hauling equipment) that are elastic (i.e., not permanent) and are not accompanied by heaving or pumping are not a problem. In addition, minor rutting under compaction equipment (roller marks) that is not accompanied with soil shear failure, is not considered a problem; however, it is an indication of a potential problem if moisture is not controlled because as repetitive loads (such as construction equipment) become excessive, or the nature of the compactive effort is not compatible with the material, could lead to a surface shear or bearing failure of the soil.

The nature of the compactive effort includes the type of compactive effort and the linear velocity (i.e., travel speed) of the compaction equipment. Minor rutting under hauling equipment, not accompanied with lateral soil shear failure, heaving, or pumping is not a problem for an isolated pass but is an indication of a potential future problem if repeated loadings are applied. Repeated loadings may result in lateral shear failure. If this occurs the damaged area must be repaired. Rutting is not only an indication of a structural problem, but also presents a functional problem that must be addressed before subbase placement can occur. These ruts will collect water, preventing proper flow to base drains, and exacerbate premature pavement distress; therefore, ruts must be properly regraded and repaired.

9.5.5 Troublesome Soils

9.5.5.1 Gap or Uniform (Open) Graded Coarse-Grained Material

Materials of this type will not bind tightly since there is not a good distribution of particle sizes. Compaction is not accomplished in the "traditional" sense of applying load to decrease air voids and compress the solid particles together. Rather, the compaction is solely accomplished by vibratory (dynamic) energy. High vibratory energy is initially needed to temporarily liquefy the particles to allow them to migrate into a denser state under the applied load. A final pass of the compactor with low vibratory energy or with the compactor in the static mode is then required to iron or smooth out the fill surface.

Particles near the surface will not obtain a tight arrangement and may be easily moved by hand. When properly placed and compacted, surface particle movement may occur under compaction or hauling equipment, but there will be no movement in the mass of material. This is not a problem but is simply the nature of the material resulting from gradation, low fines content, and no binding capability (no cohesion). The surface stability of this type of material is achieved by the confining pressure of subsequent lifts or the pavement section. If the displacement of the material is limited to the surface of the lift, this is acceptable; however, if the displacement occurs deeper in the lift, this must be repaired before placing additional material.

9.5.5.2 Fine-Grained Material

Fine-grained soils, or granular material with significant fines content (>20%), will be much more moisture sensitive than granular materials. Even when properly placed and compacted, some movement often occurs under load of compaction or hauling equipment on these types of materials. If the movement is strictly elastic compression, and is not accompanied by heaving, pumping, or rutting, this is usually not a problem. Elastic movement of fine-grained soil or granular material with high fines content, accompanied by heaving and pumping, leads to progressive shear failure of the material, and is considered an unstable condition. Repeated loading by hauling equipment can also lead to rutting. These conditions must be corrected before additional material is placed to prevent an incremental increase in the problem as the embankment or fill is constructed.

9.5.5.3 Non-Plastic Silt

Non-plastic silt is one of the most difficult material types to work with during construction due to its fine particle size and non-binding nature. It has a steep moisture-density curve and is extremely moisture sensitive. This material requires strict moisture control during placement and compaction. Due to the nature of the material, using vibratory compactors on this material likely will result in heave, pumping, rutting, and general instability. In order to avoid these issues, static (non-vibratory) padfoot compaction equipment is required to achieve the required density and reduce stability issues. The magnitude of the loading must be controlled to prevent excessive pore water pressure buildup and shear. This material is easily subject to over-compaction and shear failures; however, it can be used to construct a quality embankment or fill if care is taken during placement and compaction. Following compaction, protection and maintenance of the completed embankment is critical so that it does not become unstable due to increases in moisture content and repeated loading.

9.5.5.4 High Plasticity (Sensitive) Clay

High plasticity clay is an energy sensitive soil, meaning that minor changes in energy can result in large variations in dry density. This can result in nonuniform density in a uniform material. This condition can lead to significant future settlement or swelling of the soil. In order to avoid these conditions, compaction should be performed with a static padfoot compactor with equal pressure being exerted over the entire lift of placed material. Additionally, the moisture content must be controlled, and should be maintained to the dry side of optimum to achieve the best results.

9.5.6 "Unsuitable" versus "Unstable" Soils

Unsuitable material is soil that does not meet the material requirements of Publication 408, Section 206 for soil. It can contain too much organic matter, coal, carbonaceous materials or other objectionable matter. It can also be considered unsuitable if the material meets the specified requirements for soil but is too wet, dry or frozen to be placed. If it is considered unsuitable due to freezing or moisture issues, the material may be able to be corrected by the contractor and subsequently used. Whether an unsuitable soil can be corrected must be determined on a case-by-case basis. Improperly treated unsuitable materials will settle for many, many years. The unsuitable material is usually removed and replaced with suitable material before the embankment is constructed. If unsuitable material is proposed to be removed, plans should indicate the areas of unsuitable material, the depth of the material that must be removed and any required special treatments. If, during construction, unsuitable material is encountered unexpectedly, work in the area should be suspended until the Engineer has determined the extent of the deposit and how it should be treated. For replacement of unsuitable material for foundations, reference Publication 15 (Design Manual Part 4 (DM-4), Part B, Chapter 10, Sections 10.6.1.9P.

Unstable material is soil that meets the material specifications of Pub 408, but is damaged, too wet, or too weak for acceptance after it is placed and compacted. The strength or stability of the soil may be increased by lowering the water table and the degree of saturation of the soil. In some cases, this may be accomplished by installing temporary or permanent ditches before construction, or such material must be scarified, recompacted, dried, or removed to correct the deficiency before it can be accepted. It is more likely that unstable material can be modified and reused than unsuitable soil.

9.5.7 Compaction Control

Compaction control in the field is imperative to construct a top-quality embankment or fill. Compaction control is a general term that consists of many distinct aspects. Both engineers and construction observers should be aware of each aspect and understand how they affect compaction performance and long-term stability of the compacted materials.

Uniform compaction is as important as adequate compaction. All material in a lift must receive the same compactive effort. The type of material in each lift and a uniform lift thickness must be controlled throughout the lift, and moisture must also be controlled. Materials generally achieve better results and more stable condition when compacted on the dry side of the optimum moisture content.

Compaction equipment is another principal element to controlling compaction. The equipment must be compatible with the material type while taking special note of the roller speed. Slow roller speeds are more efficient and will require fewer passes. For vibratory compaction, if the roller speed is too fast then vibration cycles are too far apart for effective compaction. For static compaction, if the roller speed is too fast then there will be insufficient loading time for effective compaction. Details regarding acceptable compaction equipment and speed can be found in Publication 408, Sections 108.05 and 206.

Moisture and density are not the only criteria for acceptance. Stability is also a key concern. Materials exhibiting excessive yielding (heaving, pumping, or rutting) under load are not stable. Unstable material is not acceptable, and indicate compaction method problems, excessive loading, and/or excessive moisture. Drainage during construction is very important. At the end of each workday, properly grade the prepared subgrade or embankment surface to facilitate proper drainage. Seal the surface with a smooth drum static roller. Make sure the graded surface is smooth and has a proper outlet to prevent the ponding of water on top of compacted material. Do not create furrows or windrows along the edges of embankments or fills that can trap water on top of the previously placed and compacted material.

9.5.8 Acceptance of Embankments and Fills

After material has been satisfactorily compacted, is stable, and has been accepted, the contractor’s responsibility for the stability and performance of the material does not end. Any damage that occurs to previously accepted and stable material must be fixed by the contractor to an acceptable condition. This includes damage by construction traffic, other construction activities, and weather conditions.

9.5.9 Drainage and Long-Term Stability

For a properly constructed fill, long-term stability is highly dependent upon effective drainage. Adequate drainage is necessary to keep the embankment or fill from becoming saturated and maintain soil strength during service loading (traffic). Some materials are inherently difficult (but still reasonably attainable) to compact and maintain in a stable condition. An example would be a non-plastic or clean silt (high percent passing No. 200 sieve), but it has a low plasticity index (i.e., PI). Such materials exhibit a relatively steep moisture-density curve. These soils are described as moisture sensitive, meaning relatively minor changes in moisture content have a significant impact on compaction characteristics and stability. The lack of good drainage when these types of materials are present causes elevated moisture contents and can lead to pavement and slope stability failures.

Long-term stability is also dependent on the quality and uniformity of the embankment or fill materials during construction. The quality and behavior of in situ materials can vary greatly on any project. The material being placed and compaction operations/criteria must be closely observed, evaluated, and adjusted where necessary to ensure a properly compacted and stable subgrade. Both material changes and the compaction method must be considered during the construction so that appropriate adjustments can be made to maintain the quality of construction. Variable uniformity and compaction of the materials being placed can lead to weak zones in the embankment or fill that can cause instability. It is important to catch and correct these issues during construction to prevent future issues from occurring.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 10 – ACID-PRODUCING ROCK (APR)

CHAPTER 10 – TABLE OF CONTENTS

10.1 INTRODUCTION

Pennsylvania's geology includes several rock formations and structural settings that potentially contain acid-producing rock (APR). Such materials can generate acid rock drainage (ARD) when certain minerals within them are exposed to air and water. Many highway projects require significant earth disturbance, so the possibility of encountering acid-producing materials must be considered throughout the design and construction phases of Department projects.

This chapter provides guidelines, recommendations, and considerations for the investigation, testing, identification, and treatment of potential APR and ARD in highway 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 (i.e., geochemical testing, analysis, and mitigation) and criteria apply; however, sample collection techniques may differ.

10.2 UNDERSTANDING ACID-PRODUCING ROCK (APR)

A general introduction to the materials and processes involved in the generation of acid from earth materials and the consequences of acid generation is provided in the following subsections.

10.2.1 Potential Hazards of APR

APR can pose a significant effect on the environment including impacts to aquatic life due to acid drainage from highway construction projects. In addition to potential environmental hazards, exposed acid-generating earth material can result in damage to various components of highway infrastructure. Some possible highway infrastructure and environmental problems include:

- Corrosion and deterioration of concrete and steel structures and pipe inverts from acid and sulfate attack
- Deterioration of rock cut-slopes and fill-slopes due to weathering of sulfidic components
- Ground heaving of structures and pavements from oxidation of pyrite
- Toxicity to roadside vegetation
- Toxicity to aquatic life, due to an increase in the acidity of water enabling heavy metals to dissolve.
- Degradation of drinking water supplies

The environmental concerns presented by excessively acidic leachate include 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.

10.2.2 Sulfide Minerals

Although more than 200 common minerals contain sulfur, only those classified as iron sulfides are of potential concern due to their ability to oxidize and produce acid. Three iron sulfide minerals (i.e., pyrite, marcasite, and pyrrhotite) are the primary cause of acid-rock drainage (ARD).

Pyrite is the primary source of acidity in Pennsylvania rocks. Deposits containing more than 0.5% by weight pyrite and having little or no alkaline content have the potential to be significant sources of ARD when exposed during construction. Pyrite is the most common and widespread of the sulfide minerals and a common mineral in sedimentary rocks. Crystals and irregular masses of pyrite are found in small quantities in many sedimentary rocks in Pennsylvania, and some larger quantities are associated with iron deposits in coal, and pyrite is relatively common in black and dark shales in Pennsylvania's Appalachian Plateaus and Ridge and Valley physiographic provinces. The I-99 cut at Skytop on Bald Eagle Mountain in Centre County encountered a dense concentration and widespread network of pyrite veins associated with a significant zinc-lead deposit, which included a rare and highly reactive form known as "whisker" pyrite. Accessory pyrite is associated with zones of abundant flake graphite within the Precambrian Pickering Gneiss in northern Chester County and eastern Lancaster County.

Marcasite is another iron mineral and is most commonly found as tabular crystals. Marcasite most frequently occurs as replacement deposits in limestone, and often in concretions embedded in clays, marls, and shales. Pyrrhotite is commonly associated with basic igneous rocks and is found in contact metamorphic deposits, in vein deposits, and in pegmatites. Pyrrhotite has a metallic luster, is typically brownish bronze in color, and may be magnetic. A notable occurrence of pyrrhotite is the Gap nickel-copper deposit (Gap Mine) within the Pickering Gneiss in eastern Lancaster County.

There are a variety of other forms of sulfide minerals including chalcopyrite, galena, and sphalerite. These minerals are of lesser concern due to their chemical stability except when present with pyrite. When pyrite is oxidized and sufficiently lowers pH, the high levels of acidity and low pH produces additional acidity, and results in the dissolution of potentially harmful metals such as lead, zinc, copper, and nickel.

10.2.3 Sulfate Minerals

Alteration of pyrite and marcasite by oxidation results in the formation of ferrous sulfate and sulfuric acid, eventually leading to the formation of sulfate salts. These salts typically occur as white and yellow powdery coatings on exposed rock and other surfaces. During dry periods, these coatings may sequester acidity and metals. Sulfate salts readily dissolve in water to form an acid solution capable of dissolving and transporting metals.

Melanterite is one of the more common efflorescent sulfate salts and is commonly seen as a powdery white coating. Melanterite is stable only at a relative humidity greater than 59% (at

20°C/68F). Other sulfate salts include copiapite, which forms masses of yellow scale, halotrichite, and alunogen.

The dissolution of pyrite may render the host rock more porous and friable. Precipitation of efflorescent sulfate salts within the resulting pores can exacerbate the physical weathering of the host rock since the molar volume of the salts is generally much larger than the molar volume of pyrite. As the salts precipitate, the host rock pore space may be inadequate to accommodate them, and the efflorescent salts will tend to pry apart the grains of the host rock.

Sulfate minerals such as jarosite may also form in active acid sulfate soils at a pH of 3.5 or less but can remain stable for long periods of time at higher pH. Concentrations of jarosite, schwertmannite or other iron and/or aluminum sulfates, or hydroxysulfate minerals is a characteristic of a sulfuric horizon in soils. Sulfuric horizons form because of drainage (most commonly artificial drainage) and oxidation of sulfide-rich mineral or organic soil materials. Alkaline-earth sulfate salts like gypsum may also occur in weathered spoil or mine refuse; however, unlike the metal sulfate salts described above, alkaline earth sulfates like gypsum do not present a risk for acid production.

10.2.4 Potential Acid-Generating Sulfur Deposits

In Pennsylvania, potential acid-generating sulfur deposits generally fall into four broad categories:

1. Veined Rock Deposits
2. Sedimentary Rock Deposits (e.g., black shales and coals)
3. Mine Spoils (an anthropogenic deposit)
4. Acid Sulfate Soils (a supergene deposit)

The more common categories of potential acid-generating sulfur mineral deposits are described below, in generally descending order of pyrite oxidation reactivity.

10.2.4.1 Veined Rock Deposits (High Oxidation Potential)

Veined rock deposits are considered "epigenetic" sulfide mineral deposits since they are thought to have been deposited within rock pores and discontinuities by hot, mineral-rich hydrothermal fluids subsequent to the formation of the rocks enclosing them. In this publication, they will be termed hydrothermal deposits. 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. 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.

Any project site suspected of involving sulfide deposits of hydrothermal origin requires more thorough consideration and a 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)

The formation of acid from APR is a process that commonly occurs slowly in the natural environment. As a result of this process, an oxidized layer of "cap rock" overlies deposits containing concentrated sulfide minerals capable of producing significant acid. This cap rock is commonly termed Oxidized Cap Rock (OCR), or "gossan." The oxidized material produced acid under natural conditions, but at a very slow rate resulting in acid concentrations too low to result in environmental or infrastructure damage. While adequate moisture may be present, in undisturbed subsurface conditions, there is inadequate oxygen, limiting the rate of acid formation. The formation of the OCR takes many years to develop. When naturally occurring, stratified APR deposits are broken and bulked by excavation a much greater Specific Surface Area (interstitial surface area of the voids per unit volume) is created and exposed to oxygen. In the absence of alkaline to stabilize the chemistry, the addition of surface water is generally adequate to initiate rapid production.

The presence of this OCR zone can provide both an indication of the potential existence of concentrated sulfide mineralization 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 APR, by limiting excavations into the material where practical, while still meeting project objectives.

Although these Department APR 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. Existing geologic publications or past subsurface investigations are the best tools for such less common and more difficult to detect situations. In such cases, project-specific contingency actions must be developed and ready for use during construction. In such instances, qualified personnel capable of identifying such deposits should be on site during any excavations of the suspect locations. If a veined deposit is detected, mapped, or previously identified appropriate measures must also be taken to address the condition during design, and included as part of the contract items, plans and provisions.

10.2.4.2 Sedimentary Rock Deposits (Moderate Oxidation Potential)

These "syngenetic" sulfide compounds were formed entirely from the original sediments and are now disseminated throughout rock deposits. In this publication, they will be termed sedimentary 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 sedimentary APR deposits in Pennsylvania include coal and black shale deposits. In bituminous coal, pyrite occurs in the form of lenses, nodules, flakes, or fine particles. In bituminous coals, higher sulfur concentrations may be found within the following zones:

- Bottom and top of most coals,
- Toward the lateral limits of many coal beds,
- Coal directly underlying a sandstone; and
- Coals overlain by sediments deposited in brackish or marine conditions.

Additional sedimentary APR deposits include:

- Black shales, carbonaceous mudstones, and carbonaceous claystones often contain significant amounts of pyrite.
- Channel sandstones and channel sands (regular sequence of bedding structures shown in Figure 10.2.4.2-1)

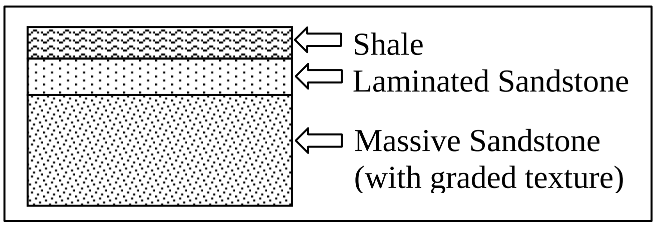


Figure 10.2.4.2-1 – Common Depositional Sequence of Channel Sandstones

Sedimentary sequences containing black shales, low amounts of calcareous material, and pyritic material should be considered especially prone to generation of ARD. Disseminated microscopic pyrite within sedimentary rocks may be evident only through geochemical analyses. Pyritic black shales within Devonian strata (Marcellus Formation and Mandata member of the Old Port Formation) produced ARD during construction of U.S. 522 west of Lewistown, Pennsylvania. Sulfur concentrations up to 17% have been measured in the Tioga Bentonite beds at the base of the Marcellus Formation.

10.2.4.3 Mine Spoils (Moderate Oxidation Potential)

Mine spoils are previously disturbed materials that 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 increasing the target Net Neutralization Potential (NNP) to 20 ppt and by excavating and managing the material in lifts no thicker than 20 feet.

10.2.4.4 Acid Sulfate Soil Deposits

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 (refer to Figure 10.2.4.4-1). In this publication, they will be termed sulfate soil deposits. Acid sulfate soils are most prevalent in areas such as brackish waters and marshes, mined materials, construction sites, dredged materials, and in regions with limited drainage and sulfate-rich water.

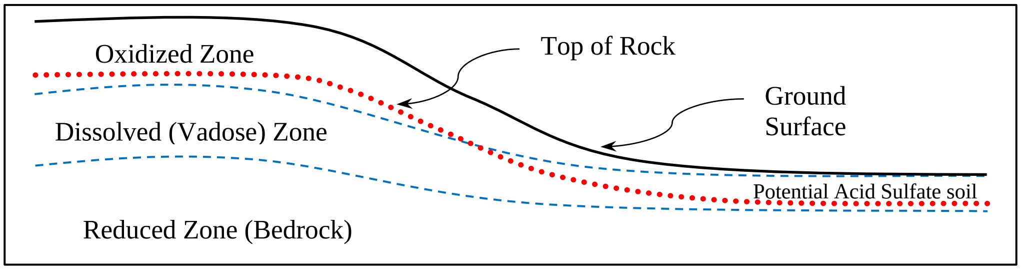


Figure 10.2.4.4-1 – Generalized Oxidation Profile

As with APR, pyrite is the primary source of inorganic acidity in acid sulfate soils, but other crystalline or amorphous iron sulfides may generate acidity as they oxidize. Left undisturbed in an aqueous, anoxic environment, acid sulfate soils are generally environmentally benign, but when exposed to oxygen by construction activities (e.g., excavation, dredging, dewatering, etc.), the sulfides break down and release sulfuric acid and soluble iron.

10.3 APR PLANNING AND MANAGEMENT

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 potential APR and/or soil exist at the site. If potential APR 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 potential APR is outlined as follows:

1. Preliminary APR Assessment
    a. Review site-specific geologic literature and mapping
    b. Perform reconnaissance field survey
    c. Contact the appropriate PA DEP Regional or District Mining Office for possible guidance on site geology and structure
2. Detailed APR Investigation
    a. Develop sampling strategy that considers the geologic structure and the presence of any faults that may occur within the proposed excavation area
    b. Perform exploratory drilling
    c. Complete background groundwater and surface water sampling and laboratory analysis
    d. Perform supplemental exploratory drilling and sampling (if needed)
    e. Perform laboratory testing of rock and soil samples
    f. Map subsurface geochemical trends relative to planned excavation

    g. Complete the Preliminary Geotechnical Engineering Report (PGER) that incorporates all information from the previous steps
3. Mitigation Design
    a. Develop final project construction earthwork plans and specifications to address the potential APR and soil deposits to be encountered. This will likely require Special Provisions and/or design drawings to detail requirements for potential APR material handling, disposal, and/or treatment.
    b. If Acid-Rock Drainage (ARD) is anticipated, and the treated discharge will be 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.
    c. Contact PA DEP Regional Office for review of the potential APR management plan.
4. Construction
    a. Monitor and document site conditions during site excavation and the potential APR treatment during construction. This will likely include surface and/or groundwater quality monitoring.
5. Post-Construction
    a. 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.

10.4 PRELIMINARY APR ASSESSMENT

The preliminary site assessment phase includes office research and field reconnaissance. Depending on the results of these activities, a detailed APR investigation may be required.

10.4.1 Literature Search

The initial step in assessing a site for potential APR is to research the applicable geologic literature and mapping that pertains to the project site. The literature review should focus particularly on the presence of geologic units (i.e., coal and dark shales) associated with acid-producing sulfide minerals, mine maps and reports of mining and ore-mineral occurrences, anomalous concentrations of metals in stream sediments and surface waters, and structural features that might control mineral occurrences.

Consult the Pennsylvania Geological Survey’s (PGS) 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 APR formations, and is a valuable first step in assessing the potential for APR. The map indicates geologic units that may have the potential to generate acid. It does not guarantee that acid generation will occur or conversely not occur in other areas not delineated on this map. Therefore, the map does not serve as a substitute for site specific subsurface investigations of rock that will be disturbed during construction activities.

Other good sources of information include the Pennsylvania Department of Environmental Protection (PA DEP), especially the District mining offices, and the United States Geological Survey (USGS). The PGS's Pennsylvania Geologic Data Exploration (PaGEODE) provides access to geology maps and publications and the ability to download and extract GIS data. The National Geologic Map Database also provides a useful portal for locating geologic maps and geologic reports.

Aerial photographs and satellite images have traditionally been used to discern linear features (e.g., fracture traces, lineaments, etc.) thought to be associated with structural features. PAMAP lidar elevation data available through the Pennsylvania Department of Conservation and Natural Resources (DCNR) allow the creation of map images using hill shading to reveal many details of faults and geologic contacts not visible in the field or in conventional aerial photographs. Such maps may be useful in locating potential areas of enriched sulfide mineralization coincident with lineaments and geologic contacts.

PADEP's PA Mine Map Atlas provides access to a vast collection of coal mine maps. Coal Mine Drainage Prediction and Pollution Prevention in Pennsylvania is a useful technical reference for projects in the Appalachian Plateaus of western Pennsylvania. Additional information can be found in PADEP's watershed reports. The Susquehanna River Basin Commission (SRBC) Mine Drainage Portal provides public access to data compiled as part of the Commission's efforts to assess and track impacts to water quality from mine drainage in the Susquehanna River basin. Additional acid mine drainage references are located at the SRBC's reports library. Consequences of acid mine drainage are discussed in PGS Water Resource Reports for Clarion, Fayette, Schuylkill, and Cambria Counties.

10.4.2 Field Reconnaissance

The site field reconnaissance, also referred to as field screening, involves walking the entire site, examining surface features for signs of potential APR. These signs may include:

- Staining by iron oxides on exposed rock surfaces; typically, in shades of yellow, orange, red, and brownish-black
- Oxide-staining in streams (orange-red streambed sediments or rock staining)
- Sterile streams (very clear water but no aquatic life)
- Sparse or unhealthy vegetation
- Outcrops of coal and/or black shale
- Outcrops having visible veins or crystals of sulfide minerals such as pyrite, galena, and sphalerite
- Evidence of past surface mining or deep mining
- Efflorescence or surface encrustations and coatings of soft powdery crystals of sulfate salts
- Low pH (<5) and/or sulfurous-smelling surface waters

Talking to residents or landowners about the site history may yield valuable insight into the characteristics of the site. The field reconnaissance may also provide an opportunity for identifying sources of acid-neutralizing materials (e.g., limestone outcrops, quarries, etc.). During the field reconnaissance, simple tests may be performed to detect the presence of carbonate materials (fizz test). Measurement of the pH and specific conductivity of surface waters may provide an indication of existing acid drainage, areas containing acid-producing materials, and areas containing sources of acid-neutralizing materials. Color may be a useful indicator for distinguishing favorable overburdens and minesoils from less favorable overburdens and minesoils with respect to acid generating potential.

10.4.2.1 Field Fizz Tests

The field fizz test is performed by adding one or two drops of 25% HCl to a fresh surface of the sample. If no reaction occurs, the fresh surface of the sample should be scraped with a knife to produce a powder (to increase the material's surface area and expose fresh powder), which is then tested by adding one or two drops of the dilute HCl. Care must be taken to ensure that the dilute HCl is reacting with the rock and not with a coating on the rock. Presence of calcium carbonate ($CaCO_3$) is indicated by visible effervescence (bubbling) or an audible "fizz." Positive test results indicate that at least 20 tons $CaCO_3$ equivalent per 1000 tons of material is present if a noticeable reaction occurs. If no fizz occurs and a sulfurous odor is detected during the field fizz test, the material should be considered potentially acid-producing.

10.4.2.2 Field pH

Measuring the pH of springs, seeps, streams, and other water bodies encountered during the field reconnaissance is recommended as these measurements may provide an early indication of the presence of acid drainage and acid-producing materials or acid-neutralizing materials. Water having a pH of 4.5 or lower is considered acid drainage. Relatively high pH values may be indicative of areas of acid-neutralizing materials. It should be kept in mind that these parameters are dynamic and may vary with season and flow conditions.

10.5 DETAILED APR INVESTIGATION

If the preliminary site assessment or any information obtained during project design indicate a likely potential for the presence of APR, then a detailed APR site investigation must be performed as part of the PGER preparation. Once the anticipated site geology and the bedrock structural orientation is defined, the locations, depths, and type of sampling will need to be determined based upon the extent of excavation anticipated for the project.

10.5.1 Planning the Subsurface Exploration Program

The planning of the subsurface investigation and its implementation includes borings and testing, drilling methods, and techniques for sample collection, logging, and storage. Table 10.5.1-1 minimum requirements for a preliminary APR site investigation (i.e., 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 APR, then additional borings and testing must be considered to further delineate and quantify the potential APR strata and the rock strata containing sources of alkalinity. Determining what is “substantial” APR must be considered within the context of the anticipated methods of mitigation and treatment of APR, and potential environmental impacts of the ARD. Any additional borings should be spaced and oriented to sample the entire stratigraphic sequence proposed for excavation and drilled sufficiently deep to provide adequate stratigraphic overlap for correlation of borings.

Table 10.5.1-1 – Minimum Preliminary Boring Requirements for APR Assessment

Type of Project Excavation	**RISK ASSESSMENT** [1, 2, 3]		
	No APR Risk based on site reconnaissance	**APR Risk** with horizontal or slightly dipping geologic deposits	**High APR Risk** or **APR 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 ≥ 300 ft.: $N_b = \frac{L}{500} + 2$ For L < 300 ft.: $N_b = 2$	For L ≥ 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 ≥ 500 ft.: $N_b = \frac{L}{500} + 1$ For L < 500 ft.: $N_b = 2$	For L ≥ 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. Local geologic conditions must be considered in the selection of individual boring locations, spacing, angles, and depths.

If geologic contacts or discontinuities are encountered that may yield varying and adverse conditions across the site, additional borings may be necessary to investigate all potential sources of APR. When preliminary results indicate the presence of potential APR, then any additional borings should be completed with specific targeting of the suspect area of cut/excavation. Results from additional testing may allow for delineation of potential APR units within the cut/excavation resulting in more efficient treatment and disposal of the potential APR.

An adequate number of borings should be obtained to enable the strata to be correlated on geologic cross-sectional plots. The number of borings required for stratigraphic overlap within the zone being characterized may vary depending on the depth of the borings, Figure 10.5.1-1, and the dip of bedding or foliation, Figure 10.5.1-2. The drilling program must attempt to adequately sample and determine the extent of all suspect strata and materials to be excavated during construction, as well as cut-surfaces (e.g., subgrades, foundations, slopes, etc.) to be left temporarily or permanently exposed. Boring depths must advance to the deepest anticipated limit of excavation, plus 5 feet.

Vertical borings are typically enough for exploration of flat-bedded geology. For inclined bedding and folding geology, if veined deposits along vertical or steeply dipping joints are suspected, or other complex structural geology, inclined borings may be necessary to sufficiently explore a sequence of deposits as shown in Figure 10.5.1-3. The location and spacing of borings are dependent upon the orientation of the anticipated sulfide deposits. Near-vertical deposits will likely require a closer boring spacing than gently dipping deposits. Angled borings are usually costlier and more difficult to obtain than vertically oriented borings. Boring inclinations up to 30º (from vertical) are usually obtainable. Inclinations greater than 45º are not typically attempted. Where hydrothermal vein deposits containing sulfide bearing minerals are of concern, a boring program should consist of multiple inclined borings completed in multiple directions to maximize the likelihood of encountering a hydrothermal vein deposit.

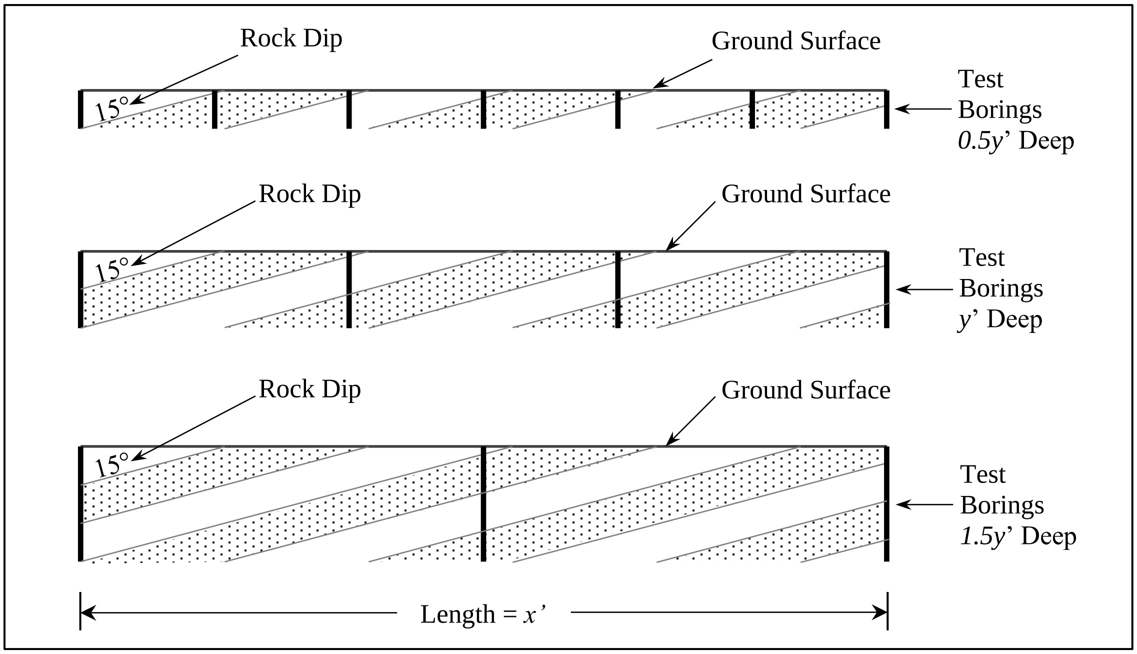


Figure 10.5.1-1 – Effect of Depth of Vertical Borings on the Number of Borings

Thoroughly characterizing the site APR conditions will help minimize the potential costs of mitigation and treatment programs, and help optimize the effectiveness of mitigation, while minimizing risks. 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 increased mitigation costs. 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.

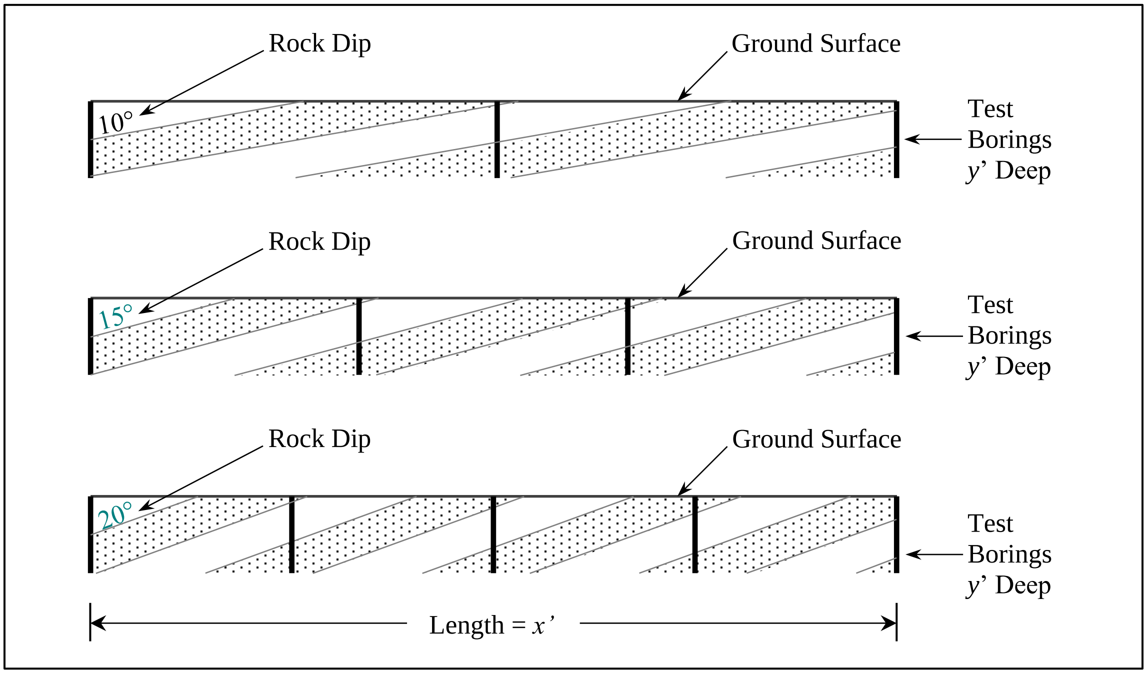


Figure 10.5.1-2 – Effect of Dip of Bedding/Foliation on the Number of Vertical Borings

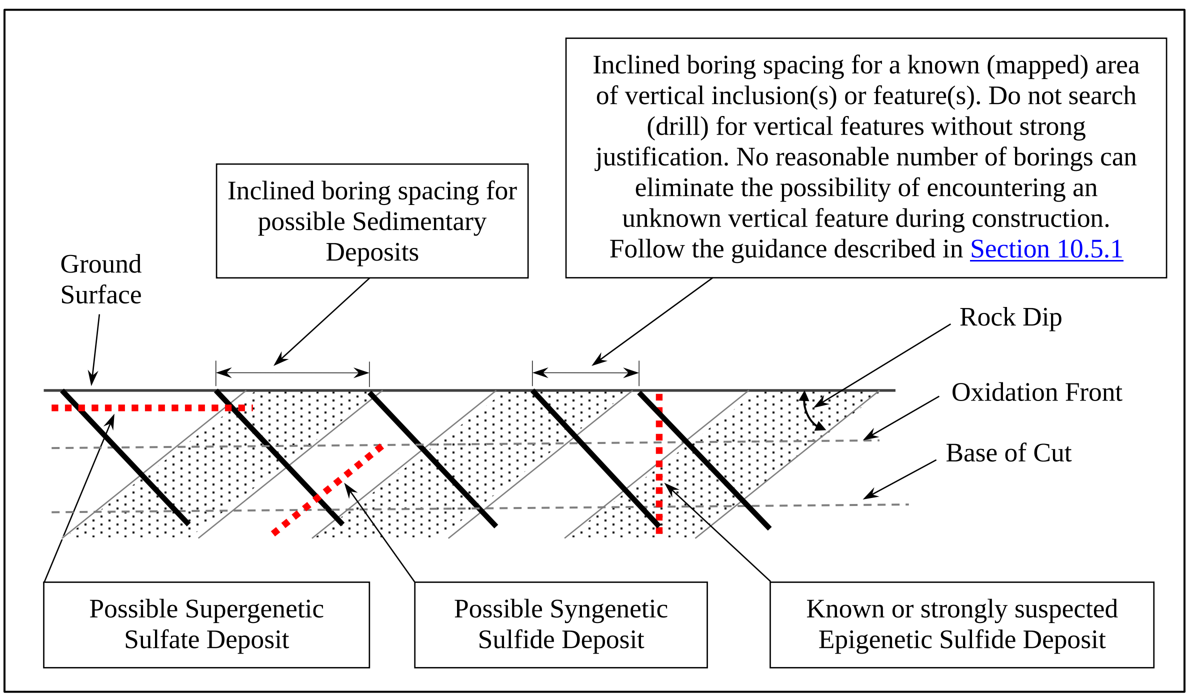


Figure 10.5.1-3 – Inclined Borings in Series to Explore Dipping Geologic Sequences

If the planned project excavations appear to have potential for developing unavoidable ARD, the receiving watercourse classification should be determined according to 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 APR, 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 experienced in treatment of ARD be included as part of the design team.

Any borings conducted as part of a detailed APR investigation must be initiated with a Subsurface Exploration Plan Submission (SEPS). 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 APR. These tasks must be completed by personnel experienced in the identification of field conditions that suggest current or potential development of APR.

10.5.2 Prevalent Alkaline-Producing Deposits in Pennsylvania

The detailed APR investigation should also focus on determining the extent of alkaline materials present, specifically materials that may be useful in mitigation of APR. 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 rock types and deposits are common sources of alkaline materials:

- Limestones
- Dolomites
- Calcareous mudstones
- Calcareous claystones
- Calcareous glacial tills
- Sandstones with calcium carbonate cement

10.5.3 Sample Collection Methods

There are three common methods of sample collection used in geochemical site investigations; core drilling, air-rotary drilling, and outcrop sampling.

10.5.3.1 Core Drilling

High-recovery core borings are the recommended method due to the ability to retrieve intact rock specimens. Important features can usually be distinguished in core samples such as fracturing, bedding plane inclinations, and identification of some sedimentary deposits of sulfide minerals. In locations where hydrothermal APR (i.e., veined deposit of sulfide minerals) is suspected, the recovery of core samples allows 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.

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) and adjusting the drill pressure and speed. Use of a triple tube core barrel system with fully longitudinally split inner core barrel is also recommended when core recovery and the quality of samples are unsatisfactory or may be of concern when drilling is known to be conducted in poor quality rock.

10.5.3.2 Air Rotary Drilling

The standard-circulation air-rotary drilling method can be 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:

- Pyrite-containing rock chips may be heavier and tend to accumulate at the bottom of the hole as lighter rock chips are preferentially removed from the hole (a process known as "salting" of the hole).
- Accurately correlating test results to specific depths and/or strata. To increase sample accuracy, drill bit advance should be halted <u>for each sampling increment (3 feet maximum)</u> and the hole blown clean before subsequent drilling.
- Rock chip samples must be collected from the collection pan only. Material is not to be taken from the accumulated return material that 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 ensure accurate representation, <u>the entire amount of material retained on the collection pan must be containerized</u>. If an excessive amount of chips is collected for one increment (i.e., greater than 15 pounds), the pan sample can be reduced according to AASTHO T248, Method B. 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 (e.g., some fills, etc.), 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.
- Logging of structural features and bedding characteristics cannot be performed.

Because of these limitations sample collection and testing from air rotary methods are not recommended for initial site characterization of potential APR materials. Its use should be limited to confirmatory sampling and testing during construction.

10.5.3.3 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 (i.e., test pits). Care must be taken not to sample material that has been exposed for a length of time and may be more weathered than the bulk of the material to be excavated during construction.

10.5.4 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 in dilute HCl solution (i.e., field “fizz” test), structural features, bedding characteristics, and fossils. High-quality digital color photographs must be taken of all boxed cores before storage or shipment to the laboratory. Photo files or prints must be included with the GER.

10.5.5 Sample Storage

All samples must 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 must 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 before storage.

10.5.6 Sample Preparation

10.5.6.1 Sample Selection and Compositing

In addition to acidic materials, the alkaline and neutral materials also need to be defined to properly assess the overall geochemistry of a site. It is important to sample all zones that might be used to stabilize and/or neutralize APR. Therefore, it is necessary to test all recovered rock core (i.e., the entire core run) within the designated sample limits. During preliminary APR testing, the maximum length of core for any one test sample is 3 feet, if subsequent APR testing is required, the test interval can be reduced to 1 foot if and/or where necessary for more refined characterization of the material. Group the tested material by sequential depth and strata. Strata thicker than 3 feet must be broken into two or more test samples. Test the 1 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 allowed under certain conditions. However, compositing must not 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 3 feet, except if a specific unit is suspected of high potential acidity, then test that specific strata alone without combining with adjacent material. Individual samples with different color, texture or fizz rating may not be combined to create a composite sample.

10.5.6.2 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 increments not exceeding 3 feet. If it is not practical to retain all the material collected for a given 3-foot, homogenize the material for that increment by thorough mixing and retain a minimum one-pound sample for testing. Each bagged sample will be crushed (if necessary) in the laboratory to allow all particles to pass the No. 60 sieve. Samples from air-rotary drilling must be air-dried before 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 sieve according to AASHTO T27. Core samples may also be split longitudinally before crushing 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 ensure. A 12-inch length of split 2-inch diameter rock core yields an adequate amount of approximately 1 to 1.5 pounds of crushed sample. 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 allowed to air dry immediately after extraction, and must be protected from rewetting, precipitation or excessive humidity until delivery to the laboratory.

10.5.7 Borehole Imaging

Borehole imaging may provide a means to observe and record continuous, high-resolution, spatially oriented, 360-degree images of a borehole wall. Such images may be used to observe and log lithology, bedding, foliation, and fractures. Computer software programs are available for processing borehole imaging data to determine borehole orientation and the orientation of features visible in the borehole wall. Borehole imaging may be useful in resolving zones of core loss, observing fracture conditions, and correlating strata on geologic cross-sectional plots. Borehole imaging is only viable in a dry borehole (i.e., above groundwater) or a borehole containing low turbidity water. While borehole imaging is an available tool to aid in the assessment of potential APR, compelling need and/or payoff should be demonstrated to justify the time, cost and effort of conducting the imaging.

10.5.8 Geophysical Exploration

Magnetite and metallic sulfides such as pyrite are the only minerals that can be detected directly using geophysical methods. During preliminary study, geophysical surveys may be useful for evaluating the presence of APR when pyrite presence is indicated by review of geologic literature or by inspection of rock specimens (from outcrops or core), but rock is poorly exposed through the proposed highway corridor. Geophysical surveys may be particularly useful in the case of steeply dipping or vertical beds, which require angled borings and a closer borehole spacing to thoroughly sample. Potentially useful geophysical methods include electrical resistivity (ER), induced polarization (IP), spectral induced polarization (SIP), self-potential (SP), and very low frequency electromagnetic (VLF-EM).

10.5.9 Groundwater and Surface Water Sampling

During the subsurface investigation, groundwater samples should be collected in completed boreholes to obtain baseline data. Boreholes should be pumped out before groundwater sampling. Pumping assures drill water is removed and the hole is allowed to refill with representative groundwater. Pumping should be conducted immediately after completion of a boring, and then allowed to refill for a minimum of 24 hours before sampling. In porous rock formations, 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 potential APR. Generally, if groundwater water has a pH less than 5.0 and any combination of the following, APR 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 μs/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.

Preconstruction water quality monitoring will provide a baseline for assessing the effect of construction activities and for design of treatment systems. Preconstruction water quality monitoring should include surface water sources, monitoring well samples, and samples from private water supply wells near proposed earth disturbances having the potential to encounter APR. More than one preconstruction sampling event may be required to assess the season variation that must be factored into the design of mitigation measures. Where APR is known or suspected, basic information should be obtained as follows:

- Field parameters such as pH, specific conductance, temperature, and dissolved oxygen (if pH is <6)

- Total metals (e.g., iron, manganese, zinc, aluminum, other metals as warranted by site-specific geology, etc.)
- Wet Chemistry, specifically alkalinity (if pH >5), acidity (if pH <5), sulfate, and hardness

Laboratory tests should be performed by a PADEP-certified lab according to standard methods for analyzing surface and groundwater samples, such as those prescribed in USEPA SW-846. Additional water quality analyses may be required to document background or baseline conditions for the design of water treatment measures when ARD is known or anticipated to occur. If drilling and testing results suggest ARD is likely to occur that would result in mitigating ARD from dewatered excavations or seepage of groundwater post construction then, constructing monitoring wells and conducting a pump test may be warranted to aid in the future design of appropriate mitigation systems.

10.6 SAMPLE TESTING TO DETERMINE THE POTENTIAL FOR APR

Tests that must be conducted to proceed with acid-base accounting are:

1. The “Fizz Rating”
2. Neutralization Potential (NP)
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.

Table 10.6-1 – Acid-Base Accounting Test Methods

Test	**Required Version/Method**	**Required Test Specimen passing the No. 60 sieve (AASHTO T27)**
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 D4239)	0.02 ounce (0.5 grams)

10.6.1 Fizz Test

While the Fizz Test is used in preliminary testing to assess if material is alkaline, it’s purpose here 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 ($CaCO_3$). Note the rate or strength of the reaction. Based upon the observed reaction, assign a "Fizz Rating" (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. The NP test does not progress far enough to produce a net zero NP. Only the initial alkaline production is measured, overestimating the total NP of the sample. To compensate for the overestimation of alkalinity and NP, a correction is performed by adding a 30% solution of hydrogen peroxide (H_2O_2) during the test process.

10.6.2.1 NP Test Procedure:

Prepare a 2-gram 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% H_2O_2 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 $CaCO_3$ 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

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 enough to neutralize the entire base present in the 2 grams of sample. A duplicate sample should be run using the next higher volume and/or concentration of acid as indicated in the table above.

10.6.2.2 NP Test Calculations

NP is calculated based on the ratio of the volume of acid to the volume of base in the blank, the volume of acid consumed, and the normality of the acid dispensed.

1. $Constant\ (C) = \frac{ml\ acid\ in\ blank}{ml\ base\ in\ blank}$
2. $ml\ acid\ consumed = ml\ acid\ added - (ml\ base\ added)(C)$
3. $NP\ = (ml\ of\ acid\ consumed)(25)(Normality\ of\ acid)$. *NP* is measured in ppt $CaCO_3$ equivalent.

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 (FeS_2) and its polymorph marcasite (FeS_2), chalcopyrite ($CuFeS_2$), 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 not well developed or standardized. 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 D4239), which are the simplest and most frequently used, and provide accurate results.

Total sulfur tends to provide a conservative estimate of the sample's acid-producing potential since the sample may contain other sulfur species (e.g., organic sulfur, alkaline-earth sulfate, etc.) whose acid-production potential is less than that of the sulfide minerals. If there is strong evidence of an excess of organic-sulfur or sulfate-sulfur significantly influencing the total sulfur concentration, additional testing for pyritic sulfur may be warranted to avoid unnecessary or excessive mitigation. For example, the presence of significant amounts of organic sulfur or gypsum may result in a relatively high total sulfur content, but the strata may have a low acid-generation potential because the sulfur is organic or in a compound of calcium sulfate instead of iron sulfide.

10.6.4 Kinetic Testing

Kinetic test method, ASTM D5744, must be considered during project design only if any of the standard ABA testing indicators are inconclusive (e.g., NNP values between zero and 20 ppt $CaCO_3$, Potential Ratio (PR) values between 1 and 2, etc.). 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 D5744 and U.S. 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 (e.g., pH, alkalinity, acidity, specific conductance, sulfate, etc.) of untreated APR fill samples and alkaline-treated APR fill. The number of kinetic tests should be based on the results of the ABA testing, site conditions, and excavation limits.

10.7 ANALYSIS AND INTERPRETATION OF TEST RESULTS

This section describes the formulas for determining Maximum Potential Acidity (MPA), Potential Ratio (PR), and Net Neutralization Potential (NNP).

10.7.1 Maximum Potential Acidity (MPA)

The MPA represents the acid generating potential of the material and is a function of the material's measured total sulfur content. Total sulfur content provides a maximum estimate of the material's acid-producing potential.

Thus, the MPA (in ppt $CaCO_3$ equivalent) is determined as follows

$$MPA = Total\ \%\ Sulfur\left(\frac{31.25\ ppt\ CaCO_3}{1\%\ Sulfur}\right)$$

10.7.2 Potential Ratio (PR)

The PR, also known as Net Potential Ratio, is the ratio between the Neutralization Potential (NP) and the MPA:

$$PR = \frac{NP}{MPA}$$

In theory, ratios greater than 1 are net alkaline, and ratios less than 1 are net acidic.

10.7.3 Net Neutralization Potential (NNP)

The NNP (in ppt $CaCO_3$ equivalent) is the difference between the NP and the Maximum Potential Acidity:

$$NNP = NP - MPA$$

In theory, sites with positive values (+) should produce alkaline water, and sites with negative values (-) should produce acidic water.

10.7.4 ABA Test Results Presented in gINT

Upon completing ABA calculations and determining the NP, %S, and NNP the results should be entered into the gINT project file. Presenting the stick boring logs along with the ABA calculations on a fence plot by elevation facilitates the identification and orientation of possible APR layers or zones within the project area.

10.7.5 Interpretation of Laboratory Test Results

Guidance on interpreting laboratory test data from overburden and rock samples collected during the subsurface investigation is discussed below. Some quantitative thresholds are provided for each test or a combination of test methods.

Table 10.7.5-1 provides guidance for interpretation of test results and analyses. Six categories of interpretation are identified. Strata that have total sulfur concentrations of more than 0.5% may generate significant acidity. Materials that have a NP greater than 30 ppt $CaCO_3$ and fizz are significant sources of alkalinity. A 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 NNP less than zero ppt $CaCO_3$ will most certainly produce acidic conditions, and an NNP value of 20 ppt $CaCO_3$ or higher will likely produce alkaline conditions. Material having NNP values between 0 and 20 ppt $CaCO_3$ may produce acidity or alkalinity; however, a NNP value of 12 ppt $CaCO_3$ or higher is considered favorable for alkaline conditions to prevail. Material with an NNP value of 30 ppt is confidently expected to generate alkalinity. As indicated in Section 10.6.4, kinetic testing should be considered for materials having NNP values between zero and 20 ppt $CaCO_3$ and PR values between 1 and 2.

Table 10.7.5-1 – Interpretation of ABA Results

NNP (ppt)	PR	Fizz Rating	NP (ppt)	% Total Sulfur	Interpretation	Recommended Action
-	-	≥ 1	> 30	-	Significant source of alkalinity	No treatment/disposal of material required and can potentially be used for APR mitigation design
> 30	-	-	-	-	Very likely to produce alkalinity	No special treatment/disposal of material required
20 – 30	> 2	-	-	-	Likely to produce alkalinity	
0 – 20	1 – 2	-	-	-	May produce acidity or alkalinity	Treatment may or may not be required
< 0	< 1	-	-	-	Likely to produce acidity	Treatment and disposal of material required
-	-	-	-	> 0.5	May generate significant acidity	

The initial target NNP level in the ABA calculations is set between 12 and 20 ppt $CaCO_3$, with a subsequent factor of safety (FS) also applied (FS= 2.0). This equates to a final target NNP value between 24 and 40 ppt $CaCO_3$. The additional quantity of supplemental alkaline material (SAM) incorporated due to applying the FS helps to ensure that adequate alkaline material is available within the amended fill. The FS guards against the effects of imperfect mixing/blending, accounts for some variability in the rock material that may occur during construction, and possible excess acidity due to the generation of carbonic acid as a result of incomplete exsolving of carbon dioxide gas during the acid neutralization process. Refer to Acid Base Accounting – Worked Examples webpage for specific worked examples

Laboratory test data are one component of the assessment of the acid-producing potential of rock material. Other considerations that may factor into the APR assessment include historical performance, preconstruction water quality, stratigraphy and lithology, groundwater flow systems, weathering effects, and the proposed construction sequencing. Prediction of APR should be based on an integrated evaluation of all these factors.

10.8 ACID DRAINAGE MITIGATION METHODS

The goal of any excavation in potential APR is to avoid the generation of ARD, and if proper mitigation techniques have been followed, the production of ARD is probably 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. Most ARD treatment systems involve alkalinity addition and metal precipitation. ARD treatment systems fall into two categories—active and passive, which are discussed below. Employing both active treatment (for major discharges) and passive treatment (for less contaminated discharges) may be appropriate in some cases. Active treatments may be required in the early stages of oxidation when acid production rates are highest, while passive treatment may be enough as the acid production rate slows along with the rate of oxidation of the surface rock.

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 (PE) 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 PA DEP.

10.8.1 Active Treatment

Active treatment systems are typically used to treat higher levels of acidity and dissolved metals and generally involve dosing the ARD with an alkaline reagent and collecting the flocculates in ponds. Reagents used generally consist of calcium compounds such as hydrated lime [calcium hydroxide, $Ca(OH)_2$], limestone (calcium carbonate, $CaCO_3$), and quick lime (calcium oxide, CaO) and sodium compounds such as sodium hydroxide (caustic soda, NaOH) and soda ash (sodium carbonate, Na_2CO_3). The type(s) of system required will depend upon the chemistry of the ARD, site and permit requirements, and treatment costs.

When active treatment compounds are dissolved in water, heat is generated, so temperature effects may need to be considered, especially in the case of trout streams of marginal quality. Trout reproduction requires stream temperatures in the range of 40 to 50˚F for a few months, and the continued survival of healthy trout requires stream temperatures to not exceed about 70˚F. 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 that require disposal on a regular basis. Operation of active treatment systems often requires access to replenish chemical supplies, perform maintenance on equipment (e.g., power supply, pumps, filters, etc.), and collect byproducts for disposal. Active treatment systems are considered reliable and effective if regularly controlled and maintained.

10.8.2 Passive Treatment

Passive treatment systems rely on naturally occurring chemical and biological processes. Advantages of this system are less labor intensive, generally require only occasional maintenance, do not require use of hazardous chemicals, effective in removal of metals, and takes advantage of naturally occurring chemical and biological processes. Types of passive treatment systems include Anoxic Limestone Drains (ALDs), Open Limestone Channels (OLCs), limestone settling ponds, Sulfate Reducing Bioreactors (SRBs), and wetlands. Disadvantages include not working well for large flows (> 500 gpm), needs a large treatment area, 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, flow rate, local topography, and characteristics of 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 Mining Operations, Technical Guidance Document 563-0300-010. In an evaluation of 116 passive treatment systems for acid mine drainage, Skousen and Ziemkiewicz (2005) found most passive systems were effective for more than five years; however, performance varied greatly within each system type.

10.8.2.1 Wetlands

The way that 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
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. Skousen and Ziemkiewicz (2005) found optimal aerobic wetland performance is expected when the pH is 6.0 or above and when the water is net alkaline. Net alkalinity may need to be imparted to acidic water by means of conventional treatment by pre-treatment using other passive systems before the water enters the aerobic wetland. Aerobic wetlands or settling basins must be designed to retain the amount of solids expected to be removed, either through periodic cleaning or through the freeboards, or through a combination of both.

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.

10.8.2.2 Limestone Settling Ponds

Water is collected and retained to allow suspended solids enough time to precipitate or "drop out" of water. Iron settles to the bottom of the pond and collects there, which after time needs to be removed. For ARD high in dissolved iron, the large surface area of a settling pond allows atmospheric oxygen to dissolve into the acidic water, which reacts with the iron to form iron oxide (yellowboy), and then subsequently settles out.

10.8.2.3 Anoxic Limestone Drains (ALDs)

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. The effluent of the ALDs must be directed into an aerobic wetland or settling basin to remove the metals.

10.8.2.4 Open Limestone Channels (OLCs)

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.

10.8.2.5 Sulfate-Reducing Bioreactors (SRBs)

Sulfate-Reducing Bioreactors (SRBs) direct water into an anoxic chamber containing organic matter and sulfate-reducing bacteria, which produce sulfides for metal sulfide precipitation, while generating alkalinity. SRBs are applicable to drainage with high acidity and a wide range of metals and offer the advantages of the ability to work in cold environments, handle high flow rates of mildly affected ARD in moderate acreage footprints, treat low pH acid drainage with a wide range of metals and anions, and accept acid drainage containing dissolved aluminum without clogging with hydroxide sludge. Design of the organic substrate used in SRBs is complicated by the wide variability of organic materials that may be locally available at a reasonable cost. Leachate collected from the Engineered Rock Placement Area (ERPA) used to store APR from Skytop is treated using a passive-treatment train, which includes a SRB, a limestone polish, and aerobic settling ponds.

10.9 ACID-PRODUCING ROCK AND SOIL MITIGATION METHODS

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. The three general “tools” available for designing an effective strategy to prevent acid production are:

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 (i.e., alkaline addition) as the primary method used in most cases. Minimizing availability of oxygen and moisture are viewed as supplemental mitigation strategies. One or more of these tools are incorporated in each of the methods discussed in detail in the following subsections.

10.9.1 Alkaline Addition

A straightforward and common approach to preventing acid generation is to add enough alkaline material to maintain an alkaline environment and neutralize all potential acidity. It is important to note that the primary purpose of alkaline addition is to maintain an alkaline environment 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 ($CaMg(CO_3)_2$), with a less common source being calcium hydroxide ($Ca(OH)_2$) applied as a liquid slurry.

As discussed, to ensure enough buffering, and since the acid reaction occurs at a greater rate than alkalinity production, a FS must be applied to the alkaline addition. A minimum FS of 2.0 must be applied to the alkaline addition. The alkaline addition is determined 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; therefore, specified alkaline addition rates for proposed rock excavations may have to be adjusted accordingly. 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, <u>and the zone can be handled separately during excavation</u>, then 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 those zones. When considering zone or layer specific alkaline addition rates, the practicality of material management during construction operations must also be considered. The specified 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 addition 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.

10.9.1.1 The Importance of SAM Particle Size

A factor that must be considered in alkaline addition is the particle size of both the potential APR and the added alkaline material. When possible and practical, the particle size of the excavated potential APR material should be kept to a maximum. Maximum particle size will provide minimum exposed surface area (available to oxygen and water), which will help reduce the rate of acid production. The particle size of the added alkaline material must be smaller than that of the suspected APR. The size of the alkaline particles determines their short-term and long-term "availability" or effectiveness as a source of alkalinity.

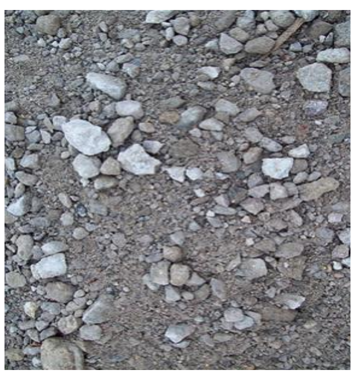
The NP of alkaline particles smaller than the No. 60 sieve is considered 100% "available", that is, effective immediately and within a period of approximately one year. Particles larger than the No. 60 sieve have less surface area and need years to weather sufficiently so the core of each particle (not just the surface) can react chemically to any surrounding acidic conditions. Alkaline particles larger than 3/8 inch 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 that needs to be recognized. While providing a readily available source of alkalinity, 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 material is to maintain an alkaline environment to inhibit acid production, it is important that the added alkaline material remain mixed uniformly through the treated mass. The No. 60 sieve 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 minimize the migration of SAM particles through open-graded excavated potential APR, a mix of well-graded limestone aggregate and agricultural lime should be used. The proportioning of coarse aggregate limestone and agricultural lime should be based on the size and gradation of the excavated potential APR. Table 10.9.1.1-1 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 potential APR. If the actual particle size of the excavated potential APR does not match the alkaline particle size distribution assumed during design, the proportioning rate and particle size distribution is adjusted as appropriate during construction.

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, potential APR 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 material 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 potential APR, the SAM mixtures should be bid as separate pay items.

Table 10.9.1.1-1 – Particle Size for Supplemental Alkaline Material (SAM) Addition

Characterization of APR Excavation	Representative Visual Gradation	Alkaline Migration Potential	Acid Production Potential	SAM Mix Gradations[1,2]
Clean Shot Rock • Minimal fines content • Large particle size • Relatively uniform particle size • Angular to Blocky fragments • Very competent		High	Low	**Coarse SAM** 65% Limestone 2A 35% PA Fine Ag-Lime OR A blended SAM meeting the following gradation: Passing 2”: 100% Passing No.4: 43-62% Passing No.60: 47-70%
Dirty Shot Rock • Significant fines • Generally well-graded • Sub-angular to angular fragments • Wide range in competency		Moderate	Moderate	**Standard SAM** 25% Limestone 2A 75% PA Fine Ag-Lime OR A blended SAM meeting the following gradation: Passing 2”: 100% Passing No.4: 62-75% Passing No.60: 70-85%
Highly Fragmented Rock • High fines content • Small particle size • Well-graded to poorly-graded • Variable particle shape • Sub-angular to flat elongated		Low	High	**Fine SAM** 100% PA Fine Ag-Lime OR A blended SAM meeting the following gradation: Passing 2”: 100% Passing No.4: 75-100% Passing No.60: 85-100%

Notes: 1. Pulverized Agricultural Limestone as specified in Publication 408, Section 804.2(a)1

2. The SAM must be thoroughly blended with the APR before compaction. SAM is not to be placed and compacted in separate layers or lifts.

As detailed in the Acid Base Accounting – Worked Examples webpage, the calculated amount of active $CaCO_3$ required to neutralize the potential APR is doubled by applying a FS of 2.0. Using a FS of 2.0 helps to ensure adequate alkalinity is available. The FS of 2.0 also helps to offset the effects of imperfect mixing, the lower alkalinity availability of the coarse aggregate limestone particles, the natural variability of rock formations, and possible excess acidity due to the generation of carbonic acid as a result of incomplete exsolving of carbon dioxide gas during the acid neutralization process. Added to the calculated FS is a built-in safety by the larger particle size of the potential APR, which limits the rate of acid production. Just as the larger particle size of the added alkaline material limits the availability of alkalinity due to reduced surface area, the large particle size of potential APR 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 FS should be established to ensure adequate alkalinity is available. Therefore, the proportion of fine alkaline particles (agricultural lime) comprises 75% 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 should be 35% due to the lower potential for acid production.

10.9.1.2 The Importance of SAM Mixing

The SAM must be thoroughly mixed with the suspected APR, to ensure proper distribution and alkalinity 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 material to acid-producing fill.
2. Determine capacity (by weight) of project haul trucks.
3. Determine necessary quantity of imported alkaline material to be added to each truck load.
4. Load haul truck with excavated acid-producing material.
5. Load the pre-determined amount of imported 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.

10.9.2 Encapsulation

All fills of potential APR or sulfate soils must be encapsulated to isolate the acid-producing material, as much as practical, from oxygen and water. All practical efforts should be made to keep the acid-producing 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 potential APR to both oxygen and water. In areas where subsurface structures are required (e.g., drainage pipes, inlets, sign foundations, guiderail posts, etc.) the capping soil must extend a minimum depth of 3 feet below any such structure.

If the gradation of the potential APR/SAM composite fill is significantly different than the foundation soil then a layer of Class-4, Type-A geotextile should be placed before 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 encapsulated material must consist of low permeability, fine-grained soil meeting one of the following criteria:

- Soil with Fines content $\geq$ 20% (min. 20% passing the No. 200 sieve), and a Plasticity Index $\geq$ 6 (min. PI = 6 for fraction passing No. 40 sieve).
- Soil, as specified in Publication 408, Section 206.2(a)1.a, and having a measured permeability no greater than 5×10^{-6} cm/s, per ASTM D5084.

Place encapsulating material as specified in Publication 408, Section 206.3(b). Soil must be underlain by a Class-4, Type-A geotextile to prevent migration and loss of the soil into the rock fill. When the potential APR fill is very coarse, angular and/or open-graded, the potential APR 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 potential APR fill. The slope of any encapsulated potential APR 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 of a fill with a rock core, the risks associated with a surficial slide or erosion of an encapsulated potential APR fill dictate a flatter 2.5H:1V slope.

If a geomembrane is incorporated into the encapsulation of the APR, the design should consider the following items:

- The site should be relatively level; placing the encapsulation on a side hill is not advisable.
- A good, properly designed anchoring system for geomembrane installation in recommended; windy days should be avoided.
- The floor of the encapsulation area should be placed on about a 0.5% to 1.0% grade, draining to an open end to provide for gravity flow of infiltrating surface precipitation during placement of the APR.
- If used as part of a roadway embankment, the amended APR should be placed in maximum 2-foot lifts and compacted according to standard specifications; if the APR is shale or phyllite, then lift thickness should be from 8 to 12 inches.
- Monitoring of the leachate is required.

10.9.3 Submergence

The permanent submergence of APR in water, is a form of encapsulation that limits the necessary oxygen needed for acid production (even though in the presence of ample moisture).

This method is most effective when the dissolved oxygen (DO) concentration of the groundwater does not exceed 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.

10.9.4 Additional Considerations for Treatment and Mitigation

While the above methods are effective, the preferred option is to avoid excavating or exposing potential APR, when possible and practical, by alignment or grade adjustments. When this is not practical, treatment and mitigation is typically necessary.

The development of an APR management plan should evaluate the range of potential solutions. The design process should be guided by sensible goals and should result in a mitigation plan that achieves 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.
- Employ mitigation strategies that balance capital costs with operation and maintenance costs.

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 are 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 must 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 that the water is located.
- Conditions of the necessary transportation network for hauling potential APR material (weight restrictions, detour requirements, etc.)
- Possible public acceptance or opposition of the potential APR management plan.

Potential mitigation factors must consider both the material being excavated and any open (temporary and permanent) excavations or cuts. 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.

10.9.5 Treatment of Excavated Materials

Handling excavated material that is identified as potentially acid-producing is primarily a material management issue. Identified potential APR or sulfate soils should be "treated" as they are excavated. The goal is to prevent the acid reaction from starting and getting established. Since 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.

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 potential APR material can usually be readily separated from overlying and underlying deposits (as in surface coal mining).

The potential APR material must be adequately described so that the appropriate personnel on the project can identify the material, and the required handling and treatment procedures can be applied. The description must include the anticipated depth(s) and physical characteristics of the suspect material(s). Identification during excavation is a key 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.

10.9.5.1 Excavated Rock

Treatment of excavated potential APR involves all three of the primary methods discussed in Section 10.9. Reference Figure 10.9.5.1-1 for a typical detail indicating minimum requirements for a full-embankment section containing potential APR fill. First, SAM is added in previously determined amounts according to acid-base accounting procedures. The SAM is provided in the appropriate gradation and thoroughly mixed as specified in the guidelines discussed in Section 10.9.1.1 and 10.9.1.2.

The treated potential APR is encapsulated following the requirements in Section 10.9.2 and 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. When the potential APR fill is very coarse, angular and/or open-graded, the potential APR must be choked-off with granular material to prevent puncture of the geotextile.

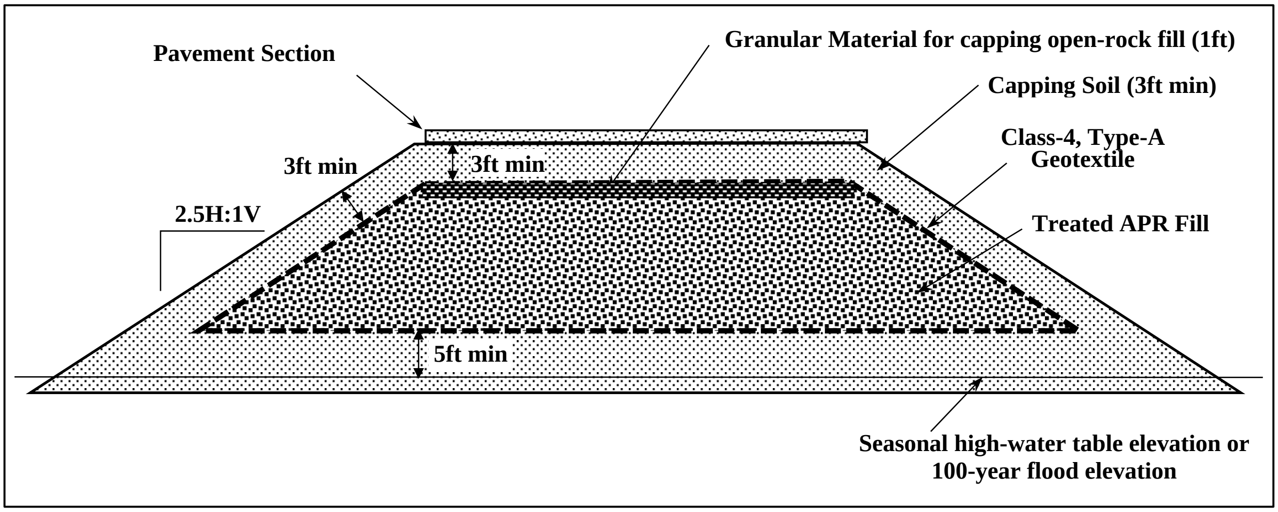


Figure 10.9.5.1-1 – Typical Section: Minimum Requirements for Treated APR Fill

10.9.5.2 Excavated Soil

Excavated sulfate soils are treated like potential APR by alkaline addition and encapsulation, except sulfate acid-producing soils present two specific problems not encountered with potential APR. The first is the rate of acid production. Because of the small particle size, acid production can initiate and progress 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, etc.). 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 sulfide 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 allowed to season, allowing the neutralization reaction and expansion to complete before 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 (i.e., alkaline addition and encapsulation) and disposed of in waste areas where no structures are planned.

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 a FS of 2.0 must be applied to the alkaline addition rates. Sulfate soils can also be treated by submergence; however, this is considered a higher-risk approach since it is likely that acid production will likely already have initiated before permanent placement below water. Treatment would still be necessary before placement and leaching of heavy metals into the groundwater may still be a potential problem.

10.9.6 Treatment of Excavated Rock Surfaces and Rock Cuts

Exposed excavations and rock cut slopes with potential APR 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 APR present in rock cuts. However, one significant difference between the open-excavation and the material excavated is the amount of 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 to further deny oxygen and moisture.

There are several approaches for treatment and mitigation of potential APR 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 and specific differences; therefore, they will be discussed individually in the following subsections.

Design and construct the rock excavations and cut slopes according to one of the Conditions (A through D) indicated in Table 10.9.6-1 and discussed in detail below. The method 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 potential APR. These include:

- Is a rock cut slope in potential APR avoidable? Have all possible alternatives been explored?

- Is the project earthwork balanced, borrow, or waste?
- Right-of-way requirements.
- Expected acid-producing potential of excavated materials and rock cut slope faces.
- Anticipated rate and duration of acid production from exposed rock cut surfaces.
- Project scheduling – excavated potential APR 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, (e.g., NPDES permit, etc.)?
- Economic considerations (cost).
- The geologic structure (jointing and bedding) and rock slope stability of cut slope in potential APR.
- Anticipated rock quality (weathering and degradation potential of rock cut faces in potential APR.
- 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, Condition 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. Condition 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 allow a soil cover. And Condition C is for cuts in much less competent or highly erodible rock, where adequate right of way exists to allow encapsulation of the cut with a soil cover. If necessary, Conditions B and C may also be applied 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 potential APR excavation.

Supplemental slope treatment options were employed on a cut in the Marcellus Shale during the construction of the U.S. 522 west of Lewistown. The Marcellus Shale exposed in the road cut was coated with a paraffin derivative, to impede air and water entry, covered with topsoil, and planted. Horizontal borings were drilled into the underlying Onondaga and Old Port Formations to intercept and drain groundwater to prevent it from contacting the Marcellus Shale.

Table 10.9.6-1 – Treatment of Excavated Rock Surfaces

Site Parameter/ Recommendation	**Condition A** (Section 10.9.6.1)	**Condition B** (Section 10.9.6.2)	**Condition C** (Section 10.9.6.3)	**Condition D** (Section 10.9.6.4)
Primary 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 APR 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	If blasting is required/allowed, follow as specified in Publication 408, Section 207			
Treatment of APR waste rock	Treat excavated APR according to Section 10.9.5.1			
Treatment Methods for APR slope face	**Direct Weathering** - With no slope face coverage, the exposed APR face is allowed to oxidize and "burn" itself out by forming an oxidized surface layer. **Shotcrete** - If the cut slope is completed in APR 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 APR surface. - Use corrosion resistant welded wire mesh, anchors and hardware.	**Geosynthetic Facing** - Provide geosynthetic cover system on the APR slope (see Figure 10.9.6.2-1. System to include combination of geotextiles, geomembrane and geocell. - To ensure 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. - Ballast geocell with coarse calcareous aggregate.	**Capping Soil** - Provide 4 feet (min.) of soil covering for slopes according to Figures 10.9.6.3-1 and 10.9.6.3-2. - Cut-slope benching required if slope steeper than 3H:1V.	**Capping Soil** - Over-excavate outcrop of APR seam. - Provide 4 feet of soil covering for slopes according to Figure 10.9.6.4-1. - Rock veneer required if slope steeper than 2H:1V.
Treatment of APR 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 OLCs, wetlands or settling ponds, and ALDs. See Section 10.8 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 drainage piping should be thermoplastic pipe (HDPE). Do not use calcareous aggregates for subsurface drainage applications.			

10.9.6.1 Condition A – Excavated Rock Slopes - 1H:1V or Steeper

Direct Weathering

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 10.2.4.1, the presence of OCR was discussed. This is APR 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 for a potential source of acidity to be converted to acid, the presence of both water and oxygen are necessary.

The concept is to use these properties in developing a plan for managing the exposed face of an APR cut slope. Once exposed, the face on the APR 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 (i.e., neutralize by alkaline addition) the acid drainage before 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 (i.e., 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 enough 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 before 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 10.8.2.

A passive treatment system typically consists of a layer of limestone through which low level acidic runoff can flow and be neutralized before release from the project is discussed in Section 10.8.2.4 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 structures, etc.)

Limestone is most effective in the smallest size practical; however, the particle size must be enough 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 (i.e., 2A or 2RC) to provide greater alkaline availability, with an adequate cover layer of coarser limestone aggregate (e.g., AASHTO No. 1, etc.) 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 10.11.2. In short, neither metal or concrete pipe fare well in acidic conditions, and the use of approved plastic pipe (e.g., HDPE, etc.) is necessary for any areas where the potential for acid exposure exists.

Shotcrete

This treatment method is applicable where a rock cut slope is completed in a sequence of durable (e.g., sandstone, etc.) and non-durable (e.g., coal, claystone, etc.) rocks. The shotcrete can be applied to APR units and non-durable units. The application prohibits non-durable units from weathering and undercutting and the oxidation of exposed APR. The design must include provisions for the use of corrosion resistant wire mesh, anchors, and associated hardware along with provisions for drainage control behind the shotcrete facing. The shotcrete is not intended to provide global stability to the rock slope. A slope stability evaluation should be completed to determine if stabilization elements are required.

10.9.6.2 Condition B – Excavated Rock Slopes - 1H:1V to 2H:1V

Geosynthetic Facing

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 (i.e., precipitation and runoff from above) are controlled. And since the encapsulation also restricts availability of oxygen, any water entering as seepage will not have enough 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 10.9.6.2-1 for a general cross-section and reference RC-78M, Slope Protection Geocell Cell and Geocell Section Details. The method 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.

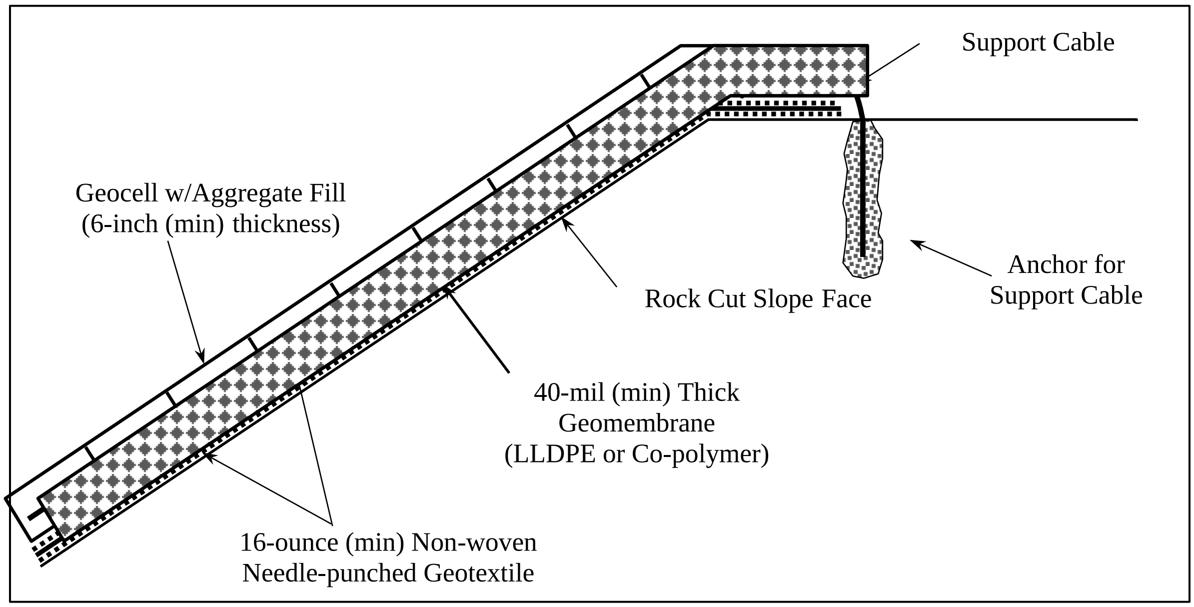


Figure 10.9.6.2-1 – Conceptual Detail for Isolation of Rock Cut with Slope Face Flatter than 1H:1V and Steeper than 2H:1V – (Condition B)

The barrier should consist of a heavyweight nonwoven needle-punched felt type geotextile, similar to a Class 4, Type-A, geotextile, but of greater thickness. Class 4, Type-A, geotextile weighs approximately 12 ounces per square yard of material (12 oz./sq. yd.). The barrier geotextile should have a minimum density of 16 ounces per square yard to provide adequate protection against puncture of 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 40-mil thickness, linear low-density polyethylene (LLDPE) or propylene/ethylene copolymer membrane. The geomembrane should be lapped top down (shingle style) and all seams field welded. A second geotextile (i.e., nonwoven, 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 material is available for immediate neutralization of the acid. The 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 procedures indicated in Condition A can also be used for flatter slopes. Cost and environmental considerations may dictate which method is the preferred solution. Also, be aware that the slope may be producing acid runoff until the geosynthetic cap can be installed. Any acidic runoff must be managed as described in Section 10.9.6.1 (Condition 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.

10.9.6.3 Condition C – Excavated Rock Slopes 2H:1V or Flatter

Capping Soil

For rock slopes with a cut face 2H:1V or flatter, an effective and relatively cost-efficient soil encapsulation approach is possible (see Figures 10.9.6.3-1 and 10.9.6.4-1). 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 Condition B, the soil encapsulation denies oxygen and restricts or limits water. Water may still reach the rock surface from seepage or infiltration; however, with adequate soil cover thickness and proper vegetation, infiltration would be minimal and dissolved oxygen, levels kept below that needed to produce acid in concentrations that would be of concern.

The slope capping material must consist of low permeability, fine grained soil meeting the following criteria:

- Soil with Fines content ≥ 20% (min. 20% passing the No. 200 sieve), and a Plasticity Index ≥ 3 (min. PI = 3 for fraction passing No. 40 sieve).
- Non-plastic silts are prohibited.

Place capping material as specified in Publication 408, Section 206.3(b). A minimum of 5 feet of soil capping for slopes not exceeding 2H:1V and a minimum of 4 feet of soil capping for slopes 3H:1V or flatter should be used to ensure 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, a RECP, Type-4 should be placed and seeded with acid-tolerant grasses such as standard Seed Formula-C (acid-tolerant to pH 4) or Formula-W (acid-tolerant to pH 5) as specified in Publication 408, Section 804.2. To provide temporary erosion control until vegetation is established, the capping soil must be properly mulched in conjunction with seeding as specified in 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.

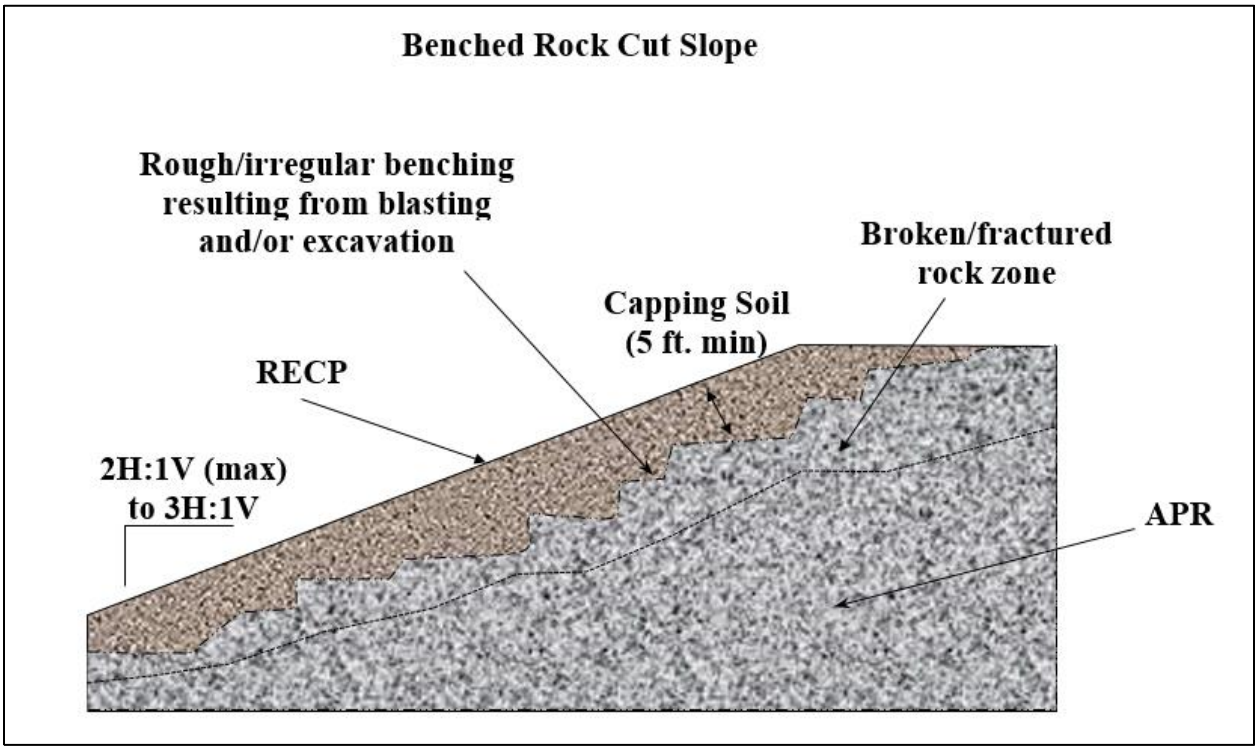


Figure 10.9.6.3-1 – Typical Section: Minimum Requirements Rock Cut with Slope Face Steeper than 3H:1V and Not Exceeding 2H:1V (Condition C)

Note that as with Condition 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 10.9.6.1. Also, the treatments discussed for both Condition A and Condition B can be applied for rock cuts with slopes 2H:1V and flatter, should specific site needs, environmental conditions, or economic circumstances dictate.

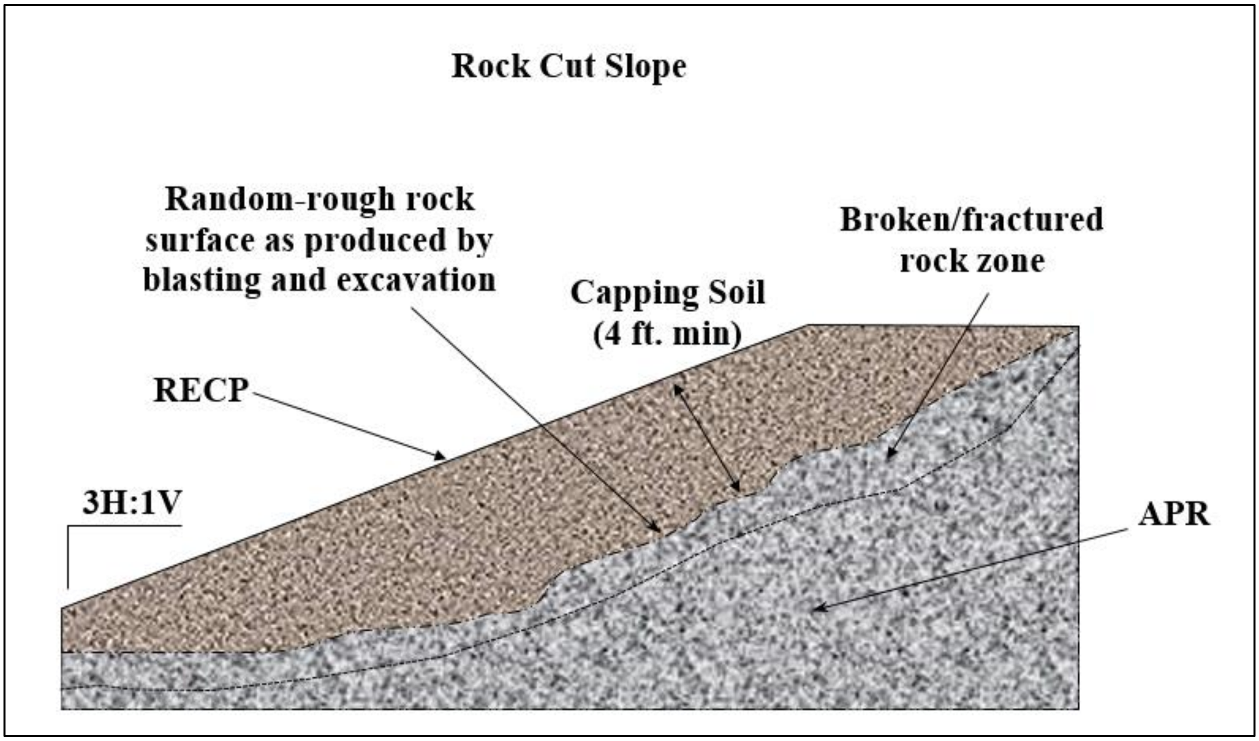


Figure 10.9.6.3-2 – Typical Section: Minimum Requirements Rock Cut with Slope Face 3H:1V or flatter (Condition C)

10.9.6.4 Condition D – Excavated Rock Face with APR Seam Treatment – Slope 1.25H:1V or Flatter

For cut slopes 1.25H:1V or flatter that contain an isolated seam of APR (i.e., 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 10.9.6.4-1. 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 Condition-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 before backfilling, similar to that specified in Section 10.9.6.3 (Condition C). If groundwater seepage from the APR seam is evident or anticipated, install an interception drain as shown in Figure 10.9.6.4-1.

Note that as with Condition B, the exposed APR 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 10.9.6.1 (Condition A).

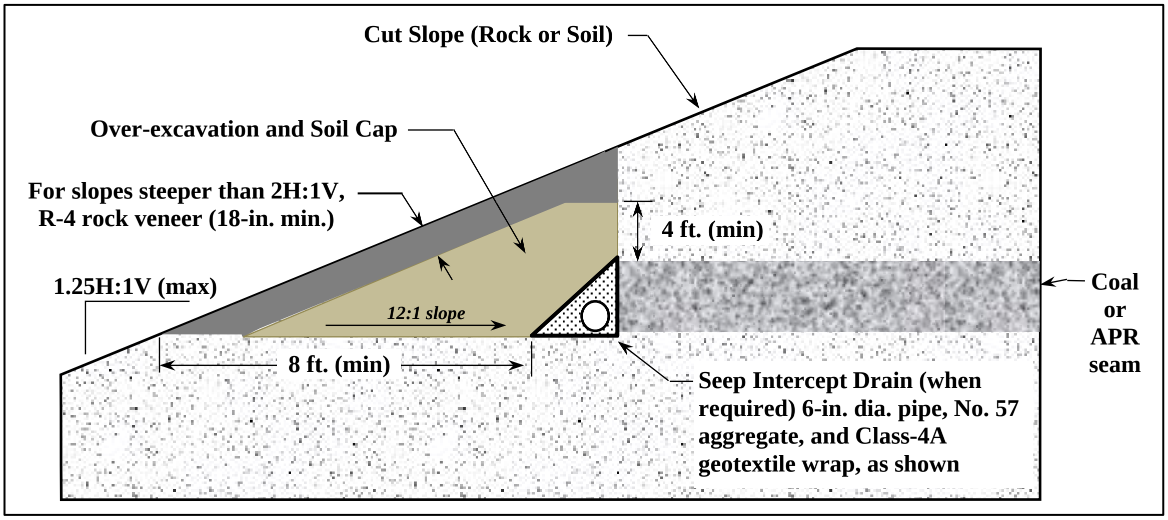


Figure 10.9.6.4-1 – Typical Section: Minimum Requirements Rock Cut with Seam Outcrop Treatment 1.25H:1V or flatter (Condition D)

10.9.7 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 10.9.6.3 (Condition C) are recommended. The same methods exist (i.e., adding alkaline, soil, compost, seeding, and mulch), and can be used and adapted as site specific needs, environmental conditions or economic circumstances dictate and/or require.

Where sustained acidic slope seepage exists or is anticipated, covering the slope face with a geosynthetic drainage composite and geomembrane may be necessary. The drainage composite functions to capture the acid seepage, which is then collected in an appropriately designed drainage system 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 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. Concentrated pyritic zones in shales of the Marcellus Formation have been identified as a cause of structural distress in central Pennsylvania. Figure 10.10-1 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 APR. Therefore, they are not specifically addressed in this policy. APR 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.

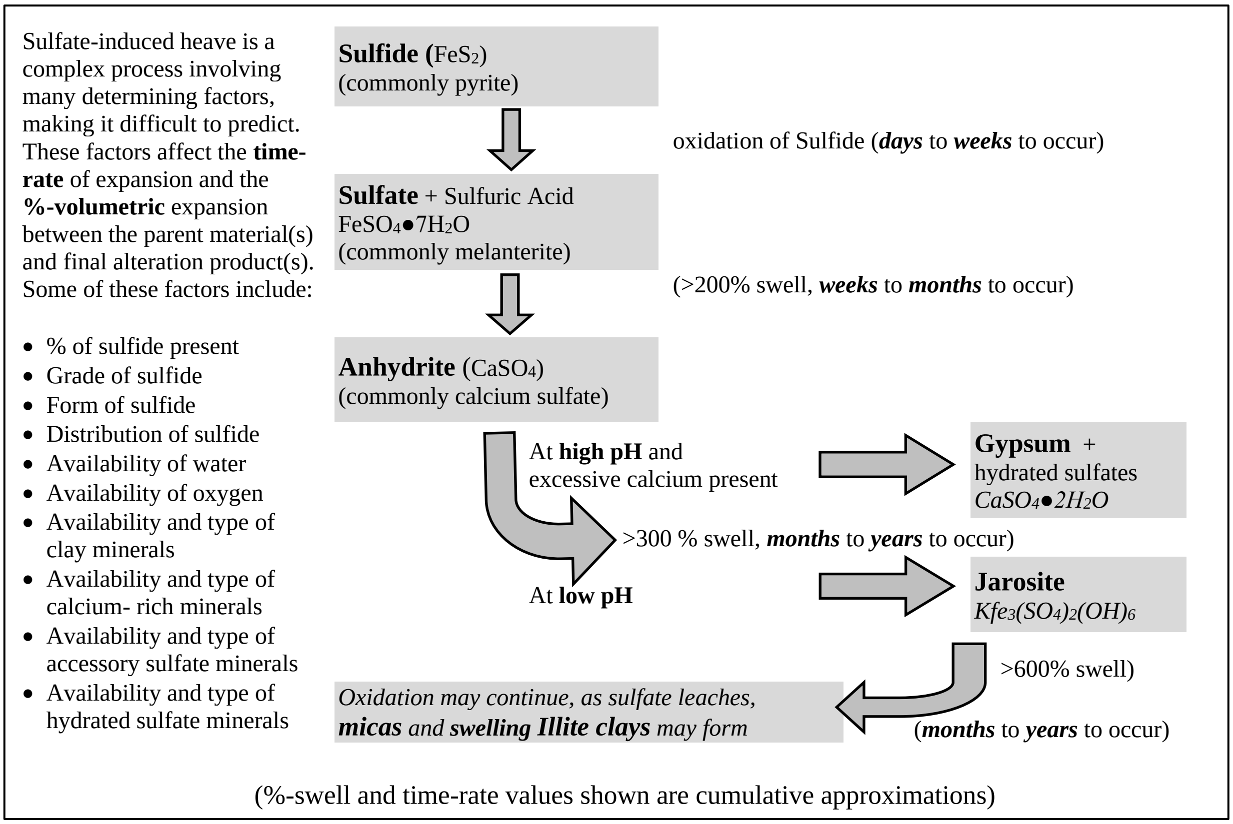


Figure 10.10-1 – Generalized Sulfide-Induced Swelling Process

When swelling shale is anticipated to be used as fill that is not encapsulated, a “mellowing” period must 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 before 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 according to test method AASHTO T290. The action points for soluble sulfate, total sulfur, and sulfide sulfur levels are listed in Table 10.10-1. It is known that for rock types other than coal, the percent-total sulfur content is not expected to be significantly different than the percent-pyritic (sulfide) sulfur content. For this reason, ASTM D4239 is the specified test method for non-coal rock types. Conversely, sulfide sulfur may be only a minor percentage of the total sulfur content in coal, and the forms of sulfur test method ASTM D2492 is needed.

Table 10.10-1 – Action Points for Soil, Rock (other than Coal) or Coal Materials Treated with $CaCO_3$ or CO for the Control of Acid Generation

<table>
<tr><th>Sulfate Sulfur in Soil

AASHTO T290</th><th>Total Sulfur in Rock

ASTM D4239</th><th>Sulfide Sulfur in Coal

ASTM D2492</th><th>Potential for 3-D Volumetric Expansion</th><th>Required Action due to Potential Swelling</th></tr>
<tr><td>0 – 2,500 ppm
(0 – 0.25%)</td><td>< 0.5%</td><td>< 0.5%</td><td>Very Low</td><td>None</td></tr>
<tr><td>2,500 – 5,000 ppm
(0.25 – 0.50%)</td><td>0.5% – 1.0%</td><td>0.5% – 1.0%</td><td>Low</td><td rowspan="2">If material is to be used as roadway or structural fill, the potential APR-SAM blend must be tested according to PTM No.130. During construction, a minimum 7-day mellowing period at +3% optimum moisture is required.</td></tr>
<tr><td>5,000 – 7,500 ppm
(0.50 – 0.75%)</td><td>1.0% – 2.0%</td><td>1.0% – 2.0%</td><td>Moderate</td></tr>
<tr><td>> 7,500 ppm
(> 0.75%)</td><td>> 2.0%</td><td>> 2.0%</td><td>High</td><td>Do not use as structural or roadway fill due to excessive swell potential</td></tr>
</table>

Materials with a “low” or “moderate” swelling potential that are intended to be used as fill must be tested according to 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. However, materials of concern due to elevated sulfide levels (above 0.5%) must be tested according to PTM No.130 for a duration of 56-days. The extended test duration is required to allow the sulfide ample time to oxidize to sulfate before 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 (e.g., below a bridge footing adjacent to a water body, etc.) 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 OTHER DESIGN CONSIDERATIONS

Other design considerations include field reconnaissance, drainage pipes, aggregates, and construction considerations involving qualified personnel and a contingency plan when APR is anticipated to be encountered during excavation.

10.11.1 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.

10.11.2 Drainage Pipe Specification

The selection of pipe material type (i.e., 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 material 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 10.11.2-1 and Publication 15M, Section 12.6.9.

Table 10.11.2-1 – Selection of Pipe Material in Acid-Corrosive Environment[1]

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	-	-	-

Notes: 1. Always reference the pipe manufacture specifications and Publication 15M, Section 12.6.9 for the various pipe material types to ensure only pipes with adequate corrosion resistance (i.e., design life) will be allowed for the project-specific conditions.

10.11.3 Aggregate Specification

When alkaline addition is required, the use of limestone aggregate with a high $CaCO_3$ content (>90%) is preferred due to its notably higher reactivity than limestone with a high

$MgCO_3$ or $CaMg(CO_3)_2$ 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.

10.12 CONSTRUCTION

During construction, the presence of APR may be indicated by encountering rocks stained with hydrated iron oxides forming the OCR over an underlying sulfide deposit or by encountering rocks containing pyrite, which may be finely disseminated and in larger localized nodules. ARD may manifest itself in the form of aquatic impacts, devegetation and stressed vegetation, staining, and the growth of efflorescent sulfate minerals.

When APR 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 design subsurface investigation. In such cases, a project-specific contingency plan should be developed by the Department and be in-place before the start of construction. The plan must address the prevention and/or mitigation of ARD during planned excavations should unexpected APR be encountered. The plan should be reviewed with PA DEP before finalizing project plans and specifications and bidding of the project.

The contingency plan should include construction phase inspection and sampling commensurate with the project size, and scope, and the APR risk. Construction phase inspection and sampling may include, but not be limited to, field inspection of excavated materials, sampling of blast borehole cuttings and disturbed materials, and water sampling. Contract specifications may require the contractor to provide field inspection and sampling during construction as deemed appropriate.

10.12.1 Field Inspection During Construction

The contingency plan or construction specifications may require a qualified Geologist/Engineer to periodically examine excavated materials and blast hole cuttings for:

- Mineralogical content (e.g., presence of sulfide minerals, etc.)
- Field fizz rating
- Presence of sulfide odor, particularly during field fizz testing

Observations during field inspection that are not consistent with design assumptions may warrant additional sampling and lab testing of the excavated material, which may in turn indicate a need to adopt additional APR mitigation measures or modify existing APR mitigation measures.

10.12.2 Rock Sampling During Construction

Sampling of cuttings and materials disturbed during construction, whether by blasting or other means, may provide verification that excavated materials were appropriately characterized with respect to APR potential during the design phase and are being managed properly during construction. In areas of potential APR, construction specifications should require sampling of cuttings from a minimum number of blast holes per blast or per volume of blasted material and a maximum sampling interval along the blast hole (or sampling interval based on lithologic change). Construction specifications should require sampling of disturbed material at a frequency based on volume of material disturbed and/or lithologic changes in areas of potential APR. Specifications should prescribe sample collection, labeling, and handling procedures. Collected samples of blast hole cuttings and disturbed materials should be analyzed for the same parameters as design phase samples (i.e., fizz rating, NP, and total sulfur). Sample testing results that are not consistent with design assumptions may indicate a need to adopt additional APR mitigation measures or modify existing APR mitigation measures.

10.12.3 Water Sampling During Construction

In areas of potential APR, construction specifications should require sampling of seeps that develop during construction or other water sources encountered but not previously sampled. Collected samples may be analyzed for pH and specific conductance. If relatively acidic water ($pH < 5$) is encountered, then additional sampling should be performed as described in Section 10.5.9. Construction specifications should also require sampling of water from storm water and sedimentation control structures to monitor for development of ARD from excavations in areas of potential APR during construction. If dewatering is required during foundation excavation, the water should be analyzed for pH and specific conductance before discharging from the excavation.

10.13 POST-CONSTRUCTION MONITORING

If ARD is detected during post-construction monitoring, additional sampling and testing to determine the source and extent of APR material contributing to the ARD will be required as well as the adoption of additional measures to mitigate the ARD. The National Mine Lands Reclamation Center of West Virginia University Publication, A Handbook Technologies for Avoidance & Remediation of AMD, provides overviews with maintenance and monitoring recommendations. Permit conditions may also govern the extent of post-construction monitoring required. The Department may develop a post-construction monitoring plan that identifies areas of concern, monitoring and sampling locations, sampling frequency, and analytical parameters.

10.13.1 Monitoring Period

In areas of APR concern, post-construction monitoring should be performed for a minimum of three years following construction to ensure that disturbed materials have been managed properly and mitigation measures are working effectively. During the first year following construction, sampling on a quarterly basis is recommended. If quarterly sampling during the first year following construction indicates no ARD is being generated, then sampling may be discontinued. If quarterly sampling indicates ARD is being generated, sampling frequency may be increased. Any active or passive treatment systems constructed should be monitored if they are in operation. Quarterly sampling is recommended during the first year following construction, and then semiannual thereafter. Treatment systems may also be subject to additional post-construction monitoring and sampling as specified in the conditions of the NPDES permit.

10.13.2 Monitoring Locations

Potential areas to be monitored include, but are not limited to, cut slopes, fill areas, embankments, areas of APR mitigation, and treatment systems. The post-construction monitoring plan should identify locations where samples of groundwater, seepage, or runoff from these areas sampling points may be collected. Potential monitoring locations may include, but are not limited to:

- Runoff from rock cuts, fill areas, and embankments, which may be directed to a holding pond, sump, vault, or other structure designed to allow collection of samples in a timely fashion following precipitation events.
- Groundwater and surface water sources upgradient and downgradient of fill zones, embankments, and APR treatment areas.
- Influent and effluent of treatment systems.
- Locations required by permits.

To the extent practical, existing and proposed monitoring points should be identified during the design phase. Existing monitoring points may be sampled preconstruction to establish a background condition. More than one round of sampling may be required to account for seasonal variation.

10.13.3 Monitoring Parameters

Post-construction water samples should be tested for the same parameters as the preconstruction water samples, which are listed in Section 10.5.9. If ARD is detected, then additional sampling may be required. Permit conditions for treatment systems may specify additional sampling parameters.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 11 – GEOSYNTHETIC REINFORCED SOIL (GRS) SLOPES

CHAPTER 11 – TABLE OF CONTENTS

11.1 INTRODUCTION

This chapter provides design requirements and guidelines for Geosynthetic Reinforced Soil (GRS) slopes on Department roadway and structure projects. Standard Drawing RC-14 provides typical details and notes for GRS slopes, and Publication 408, Section 223, must be used for construction of GRS slopes.

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 as specified in Publication 408, Section 223, and as indicated in Standard Drawing RC-14. GRS slopes greater than 40 feet in vertical height require review and approval by the Chief Geotechnical Engineer (CGE).

Stability of a GRS slope depends on the reinforced soil's 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 Department Approved Software for slope stability, or with hand calculations when appropriate (e.g., horizontal sliding, etc.). Complete the analyses by assuming the orientation of the geosynthetic tensile force is in the plane of the geosynthetic (i.e., horizontal), and use an inclination factor of zero. Assume boundary conditions to be zero at both ends of the primary geosynthetic (no anchorage) and recommend that no adhesion between the geosynthetic and the soil be applied, regardless of whether cohesion is present in the soil. Ensure minimum tensile forces are being met and check that embedment length can provide adequate pullout resistance, reference the 2009 Edition of Publication No. FHWA-NHI-10-024, GEC 011, Chapter 9. A minimum embedment length of three feet is required.

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 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 must 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. 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. 57 Coarse Aggregate as specified in Publication 408, Section 703. When using No. 57 Coarse Aggregate reinforced fill, wrap the Class 4, Type A geotextile secondary geosynthetic around the No. 57 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. 57 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 appropriately sized rock lining according to DM-4, Part A, 2019, 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 that is less than the width of the slope, the FS against failure by squeezing may be calculated by the following equation:

$$\boldsymbol{FS_{squeezing} = \frac{2c_u}{\gamma_{rf} D_s \tan\beta} + \frac{4.14c_u}{H\gamma_{rf}} \geq 1.3}$$

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 as specified in Publication 408, Section 206.2(a)1.1.a, except that it must have greater than 25% 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 as specified in Publication 408, Section 806.2(b).

The vegetated slope face provides long-term erosion control. Specify Formula C seeding mix as specified in Publication 408, Section 804. Vegetated reinforced slopes must be protected against application of herbicide treatments. Specify Bonded Fiber Matrix mulch as specified in Publication 408, Section 805.

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 must 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 before 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 must 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 before 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 must be sized as appropriate for project conditions, and must 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 before 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 as specified in Publication 408, Section 626.

Specify a Class 4, Type A geotextile backing meeting the requirements as specified in 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 on the 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.

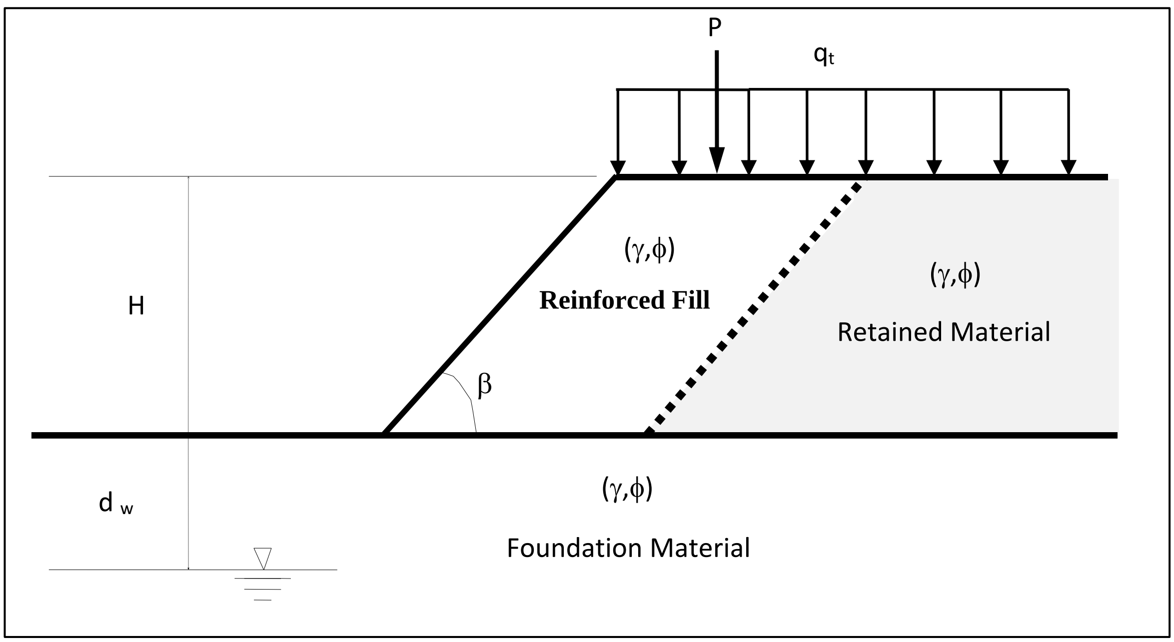


Figure 11.6.1-1 – Geometric and Loading Requirements for GRS Slope

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, (qt), 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]
External	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 FS 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.

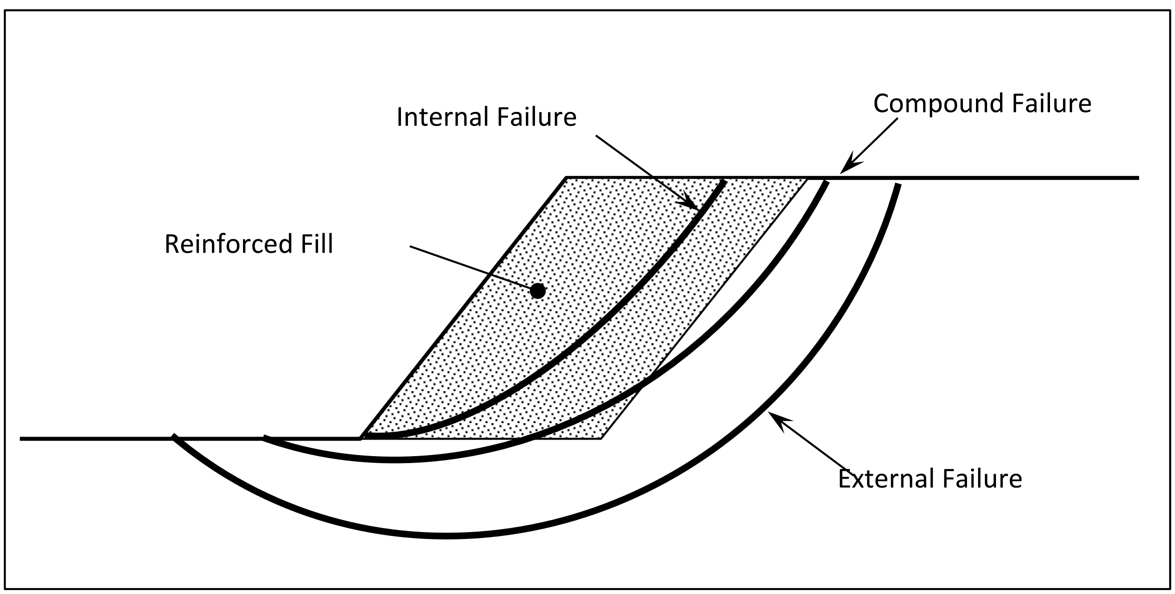


Figure 11.6.3-1 – Failure Modes

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 use an external horizontal sliding safety factor (FS) 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 D3080) or consolidated-undrained (CU) triaxial tests with pore pressure measurement (ASTM D4767). 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 (FS) 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 use an external horizontal sliding safety factor (FS) 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 D3080) or consolidated-undrained (CU) triaxial tests with pore pressure measurement (ASTM D4767). 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 (FS) 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, aa Special Provision may be necessary with included 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 on the contract documents. 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 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. For GRS slopes constructed with select granular material (2RC) from an approved Bulletin 14 source, use a design friction angle of 34 degrees for the reinforced fill. For GRS slopes constructed with AASHTO Nos. 8, 57, 67, or PennDOT No. 2A Coarse Aggregate from a Bulletin 14 approved supplier, use a design friction angle of 36 degrees for the reinforced fill.

Establishing the minimum reinforced fill internal friction angle is 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 according to AASHTO T236 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 must remain proportioned as the original sample before substitution. Shear strength testing of reinforced fill material consisting of AASHTO Nos. 8, 57, 67 Coarse Aggregate, PennDOT No. 2A Coarse Aggregate, or select granular material (2RC) is not required if the material is from a Bulletin 14 approved supplier and the 34 degree (i.e., 2RC) or 36 degree (i.e., AASHTO Nos. 8, 57, 67, and PennDOT No. 2A) friction angle is used. The maximum design internal friction angle (ϕ) value for any 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., $c = 0$).

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. 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.

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 as specified in Publication 408, Section 223.2(i).

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 according to AASHTO T289.

11.7.3.3 Gradation

The gradation of the reinforced fill material must meet one of the following:

- Publication 408, Section 206.2(a)1.1.a, Soil, except particles greater than 4 inches must be removed before compaction.
- Publication 408, Section 703.2, AASHTO No. 8, AASHTO No. 57, AASHTO No. 67 Coarse Aggregate, PennDOT No. 2A Coarse Aggregate. AASHTO No. 57s must be used in areas with wet or submerged conditions, or in areas subject to flooding and rapid drawdown.
- Publication 408, Section 703.3, Select Granular Material (2RC)
- 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

11.7.3.4 Unit Weight

The acceptable ranges of compacted unit weights for soil and aggregate reinforced fill are between 110 and 125 pcf according to PTM No. 106, Method B. Section 11.4 references the need to consider both the minimum and maximum unit weight to check for the controlling condition. 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, require AASHTO No. 57 Coarse Aggregate for reinforced fill, and use a unit weight (γtot) of 100 pcf for the coarse aggregate zone of the GRS slope. When using AASHTO No. 8 Coarse Aggregate for reinforced fill, use a minimum compacted unit weight (γtot) of 90 pcf.

11.8 GEOSYNTHETIC REQUIREMENTS

Primary geosynthetic reinforcement must consist of a geogrid meeting the requirements as specified in 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% 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 Department Approved Software. 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 must be used as the preliminary primary geosynthetic length (L_d) for the full height of the zone. The minimum length of primary geosynthetic is eight feet. The final primary geosynthetic length is determined according to Sections 11.8.3 and 11.8.5.

11.8.3 Primary Tensile Strength

Use Department Approved Software to determine the primary geosynthetic design tensile strength. Use the preliminary primary tensile length (L_d) 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 Department Approved Software 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 ensure 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 ensure 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, etc.). 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 Allowable Geosynthetic Tensile Strength

Once the optimal primary geosynthetic design strength, T_d, and length, L_d, combination have been determined for a given design section, determine the required minimum long-term allowable geosynthetic tensile strength for construction (T_L) as indicated below:

$$T_L = \frac{T_U}{RF_{0v}}, or\ T_5, whichever\ is\ less$$

where,

T_L = Long-term allowable geosynthetic tensile strength (lbs/ft)
T_U = Ultimate tensile strength, (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.

T_5 = MARV tensile strength at 5% elongation, (lbs/ft)

Calculate required T_L for both Class 1, Type A - PET and Class 1, Type B - HDPE polymers, as applicable. State the requirements for both Class 1, Type A and B uniaxial geogrids as indicated on the Contract Documents.

11.8.5 Check Primary Geosynthetic Length for Sliding Stability

Once the primary geosynthetic length (L_d) has been determined according to 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 FS_S 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 FS_S becomes the new design length (L_d) for the design section.

Determine the sliding resistance of the reinforced mass for all zones as follows:

$$FS_s = \frac{Resisting\ Force}{Sliding\ Force} = \frac{(W + P_a \sin \o_b) \tan \o_{sg}}{P_a \cos \o_b}$$

where,

$$W = \frac{1}{2} L^2 \gamma_{rf} \tan \beta \qquad \text{(for } L \tan \beta \leq H\text{)}$$

$$W = (LH - \left(\frac{H^2}{2 \tan \beta}\right)) \gamma_{rf} \qquad \text{(for } L \tan \beta > H\text{)}$$

$$P_a = \frac{1}{2} \gamma_{rm} H^2 K_a$$

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

$\o_b$ = friction angle of reinforced fill, degrees

$\o_{sg}$ = soil-geosynthetic direct shear resistance (determined according to ASTM D5321), 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/ft^3

γ_{rm} = unit weight of retained soil mass or backfill, kips/ft^3

11.8.6 Secondary Geosynthetic

All reinforced slopes require a secondary geosynthetic consisting of a Class 4, Type A geotextile, meeting the requirements as specified in 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 must be specified as a Class 4, Type A geotextile.

11.9 CONSTRUCTION RELATED CONSIDERATIONS

GRS slopes should consider placement of the geosynthetics, benching, temporary conditions including sequencing, final conditions, and transitions for slopes with surface grades during design. It is also important to note whether a GRS slope already exists near or adjacent to the proposed GRS slope, and if any necessary alteration or removal of that GRS slope is being proposed. As GRS slopes are constructed differently than conventional slopes and rely on the geosynthetic reinforcement for performance of the slope and slope stability considerations, anyone intending to alter or excavate into a GRS Slope needs to be made aware of the presence of, and risk associated with, excavating GRS slopes and altering or removing geosynthetic reinforcement.

Caution tape that is customizable detectable underground warning tape is to be placed throughout the top of the GRS slope as shown on the Standard Drawings to warn contractors or other entities against damage of the reinforcement in a GRS Slope. Likewise, constructability and slope stability concerns need to be addressed in the event of removal or alteration of a GRS Slope. In the event a GRS slope is to be removed or altered, the project specifications are to require the contractor to submit a plan for removal or alteration of the slope to the Representative for review and approval. The plan is to address both the integrity of the wall and slope stability considerations for the wall and affected portion of the project.

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 shown on the 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 as specified in Publication 408, Section 201. Foundation elevations must 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 must 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 must be trimmed to follow the finished grade.

For Type C slopes, top of geotextile wraps must be parallel to finished grade, ending at the subgrade elevation. For Type D slope face type, reduced height gabions must be used for the top of the reinforced slope, following the finished grade as close as possible. The reduced height gabions must 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 GER unless required by the DGE to be submitted under separate cover for timelier project completion. The submission must 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 (reference Section 11.11 for plan presentation)
- 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

11.11 PLANS PRESENTATION

Submit the roadway plans and cross-sections for review with the GER. The roadway plans must include, but are not limited to:

- typical cross-section for the GRS slope, including strap length, assumed excavation limits, and design notes (Figure 11.11-1)
- elevation view showing steps at the bottom and top of the GRS, including tables listing elevation and GRS height data (Figures 11.11-2 and 11.11-3)
- excavation detail that indicates excavation that is incidental to the GRS item and that is part of roadway items (Figure 11.11-4). Please note if temporary shoring is required, the excavation that is incidental to the temporary shoring item should also be indicated (Figure 11.11-5)
- the earthwork summary table must include the calculated Class 1 Excavation quantity that is paid for as specified in Pub. 408, Section 223.4, as information only (e.g., "CLASS 1 EXCAVATION INCLUDES XXX CY ACCORDING TO ITEM NO. 0223-YYYY GEOSYNTHETIC REINFORCED SOIL SLOPE TYPE ZZ"

Cross-Sections must be included at every 25 feet along the length of the GRS.

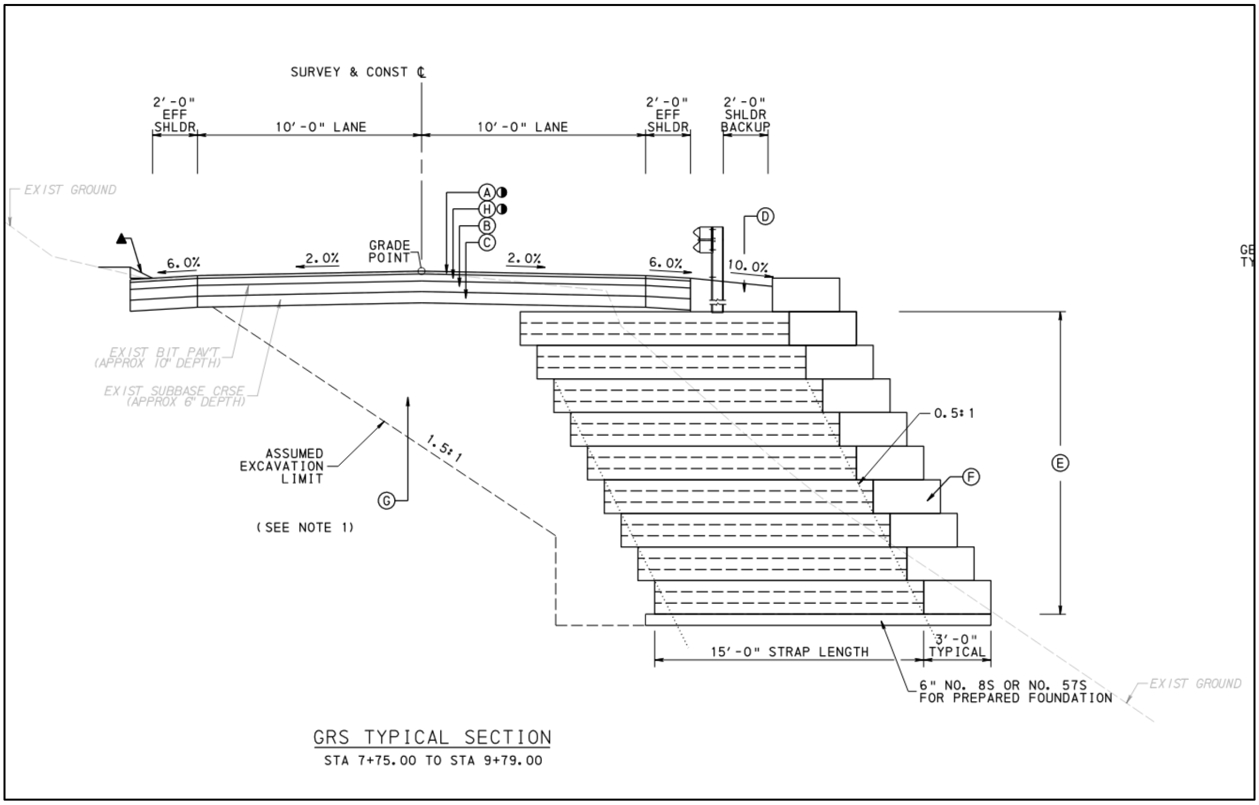


Figure 11.11-1 – GRS Typical Section

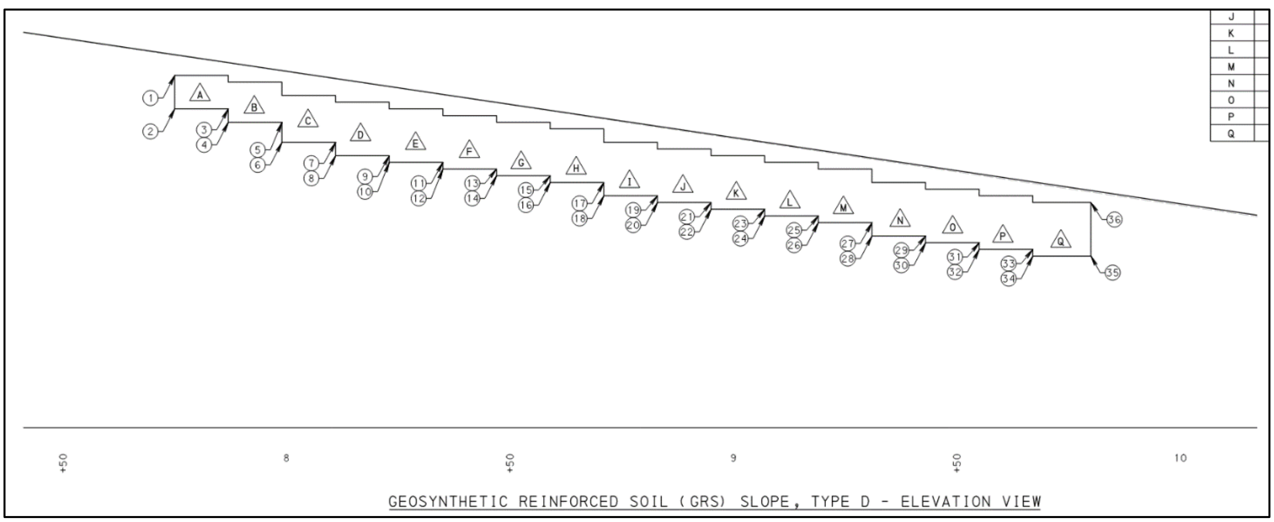


Figure 11.11-2 – GRS Elevation View

GRS SLOPE, TYPE D ELEVATION DATA		
POINT	STATION	ELEVATION
1	7+75.00	894.0′
2	7+75.00	886.5′
3	7+87.00	886.5′
4	7+87.00	883.5′
5	7+99.00	883.5′
6	7+99.00	879.0′
7	8+11.00	879.0′
8	8+11.00	876.0′
9	8+23.00	876.0′
10	8+23.00	874.5′
11	8+35.00	874.5′
12	8+35.00	873.0′
13	8+47.00	873.0′
14	8+47.00	871.5′
15	8+59.00	871.5′
16	8+59.00	870.0′
17	8+71.00	870.0′
18	8+71.00	867.0′
19	8+83.00	867.0′
20	8+83.00	865.5′
21	8+95.00	865.5′
22	8+95.00	864.0′
23	9+07.00	864.0′
24	9+07.00	862.5′
25	9+19.00	862.5′
26	9+19.00	861.0′
27	9+31.00	861.0′
28	9+31.00	858.0′
29	9+43.00	858.0′
30	9+43.00	856.5′
31	9+55.00	856.5′
32	9+55.00	855.0′
33	9+67.00	855.0′
34	9+67.00	853.5′
35	9+79.00	853.5′
36	9+79.00	865.5′

GRS HEIGHT TABLE	
GRS#	HEIGHT
A	7.5′
B	9.0′
C	10.5′
D	12.0′
E	12.0′
F	12.0′
G	12.0′
H	12.0′
I	12.0′
J	12.0′
K	12.0′
L	12.0′
M	12.0′
N	12.0′
O	12.0′
P	12.0′
Q	12.0′

Figure 11.11-3 – GRS Height and Elevation Tables

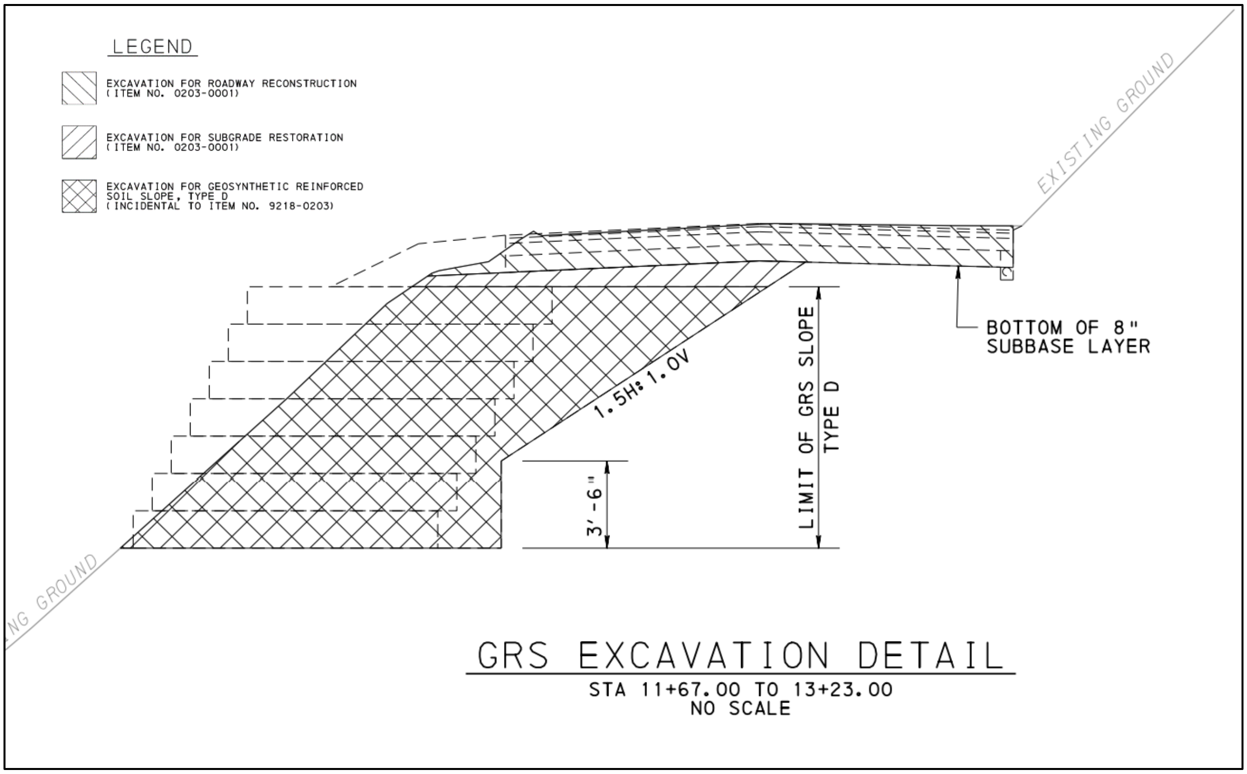


Figure 11.11-4 GRS Excavation Detail Showing Pay Items

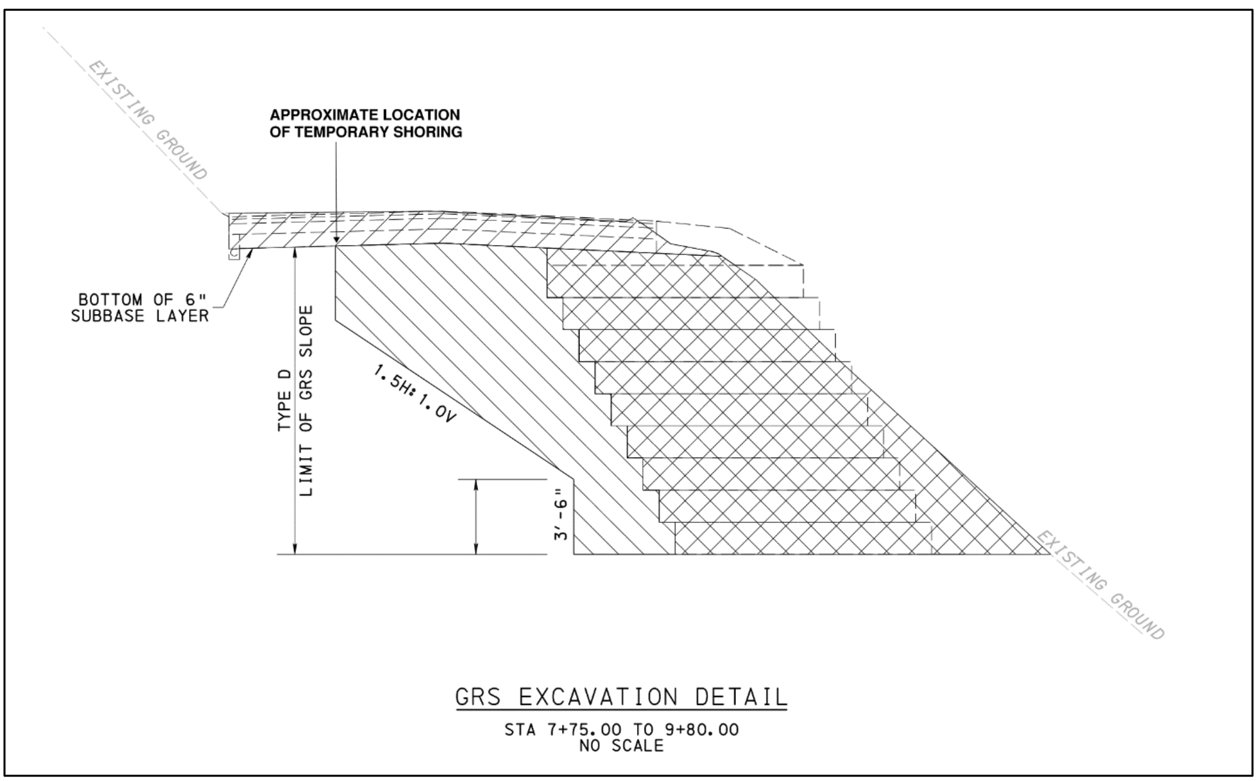


Figure 11.11-5 – GRS Excavation Detail with Temporary Shoring

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 12 – GEOCELL

CHAPTER 12 – TABLE OF CONTENTS

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 distributers 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 in^2), Type B (71.3 in^2), and Type C (187.0 in^2). 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 (i.e., 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 ensure 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 ensured. 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.

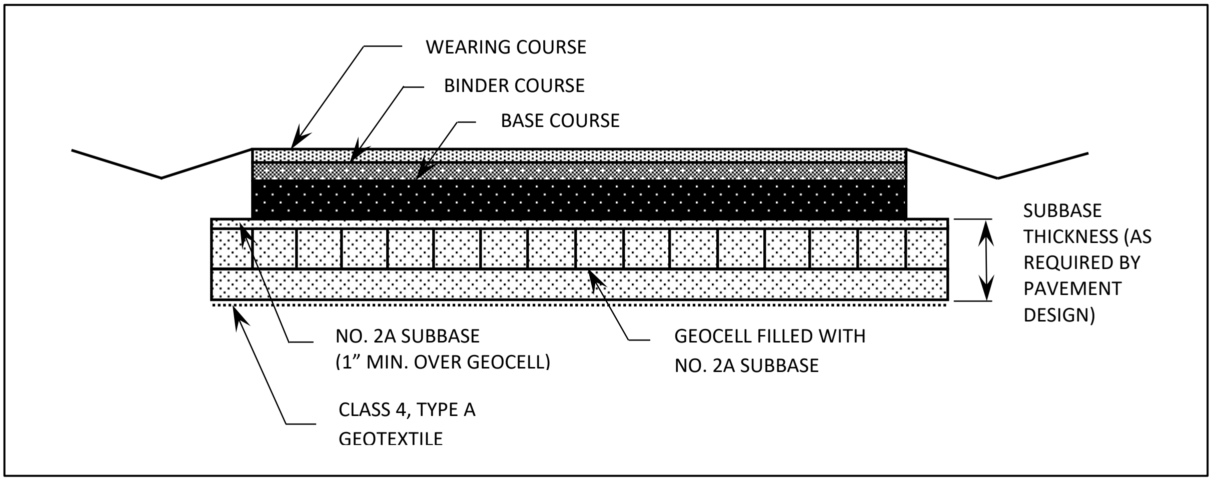


Figure 12.2-1 – Typical Section

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 in^2 (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 before paving. Type C (i.e., nominal cell area of 187 in^2) 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 before paving, or where the unstable subgrade is marginally acceptable in an unreinforced condition.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 13 – GEOFOAM

CHAPTER 13 – TABLE OF CONTENTS

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/m^3). For example, the minimum required density of EPS22 is 21.6 kg/m^3.

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/ft^3. 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 enough 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.

Like 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 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 disproportionally 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. Like 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.

Like 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 geofoam, the pressure from wheel loads is typically less than approximately 4 psi.

13.6.3.2 Dead Load

Dead load applied to EPS geofoam embankments is typically from the cover materials and pavement section that is placed on top of the geofoam. 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 geofoam results in approximately 1 psi pressure to the geofoam. Because of this, only the minimum necessary cover materials and pavement should be placed on top of EPS geofoam and, the geofoam 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 geofoam.

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 geofoam embankment designed and constructed in accordance with Publication 408, Section 219, Geofoam 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 geofoam, construction equipment is not permitted directly on the surface. Therefore, the load from construction equipment at the surface of the geofoam was estimated after placement and compaction of one lift (i.e., 6 inches) of capping material over top of the EPS geofoam. 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 geofoam, 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 Geofoam Type Based on Calculation Results

Project specific versions of the calculations discussed above must be used to select the EPS geofoam type. As previously discussed, the long-term pressure on the EPS geofoam cannot exceed the compressive resistance of the geofoam at 1% strain. The calculation performed for the Final Condition indicates that the maximum estimated long-term pressure on the surface of the EPS geofoam is approximately 7 psi based on the assumed thickness of cover material and pavement section. Based on ASTM D6817, EPS22 geofoam 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 geofoam, and 13 psi at one foot below the surface during construction. The compressive resistance of EPS22 geofoam 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 geofoam, 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 geofoam at 5% strain, EPS22 geofoam is acceptable below the one-foot layer of EPS39 geofoam.

Based on these calculations, Publication 408, Section 219, Geofoam Lightweight Fill requires that a one-foot thick layer of EPS39 geofoam be used at the surface of the geofoam block fill, and EPS22 geofoam be used for the remainder of the geofoam embankment. Note that EPS39 is not needed to cap the side slopes of the EPS22 geofoam 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 geofoam 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 geofoam will most likely not have enough 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 geofoam (i.e., EPS29 or greater as necessary) for the geofoam below the 1 foot EPS39 geofoam cap layer, or a concrete distribution slab constructed on top of the EPS39 geofoam. Depending upon the thickness of fill above the geofoam, the 1-foot capping layer may also require a higher-grade EPS geofoam. 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 geofoam 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 geofoam.

It is worthwhile restating that the compressive resistance of EPS geofoam increases as the density increases, and the cost of the EPS geofoam 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 geofoam 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 enough 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, like 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 like 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 possibly 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 enough 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.

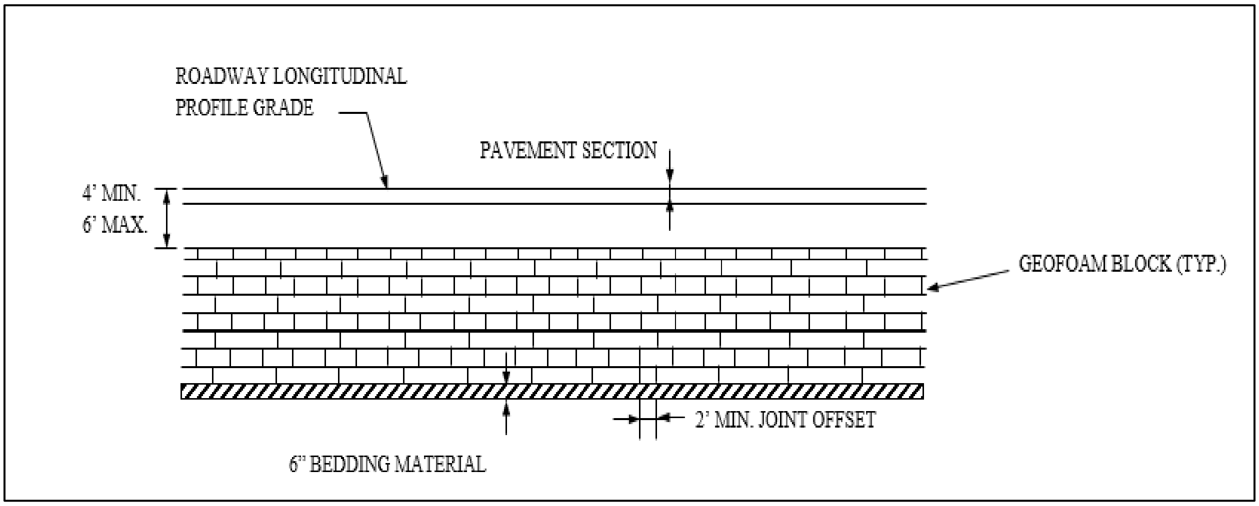


Figure 13.7-1 – Typical Longitudinal Profile View

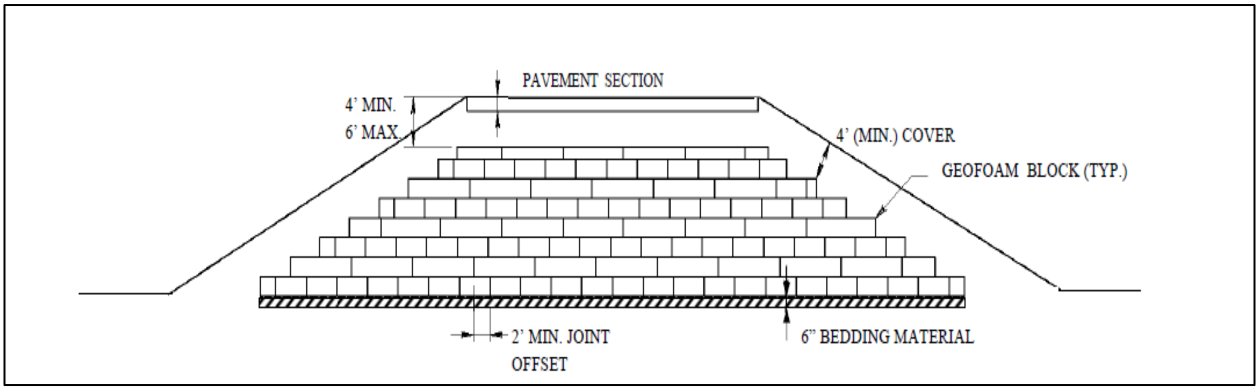


Figure 13.7-2 – Typical Embankment Section

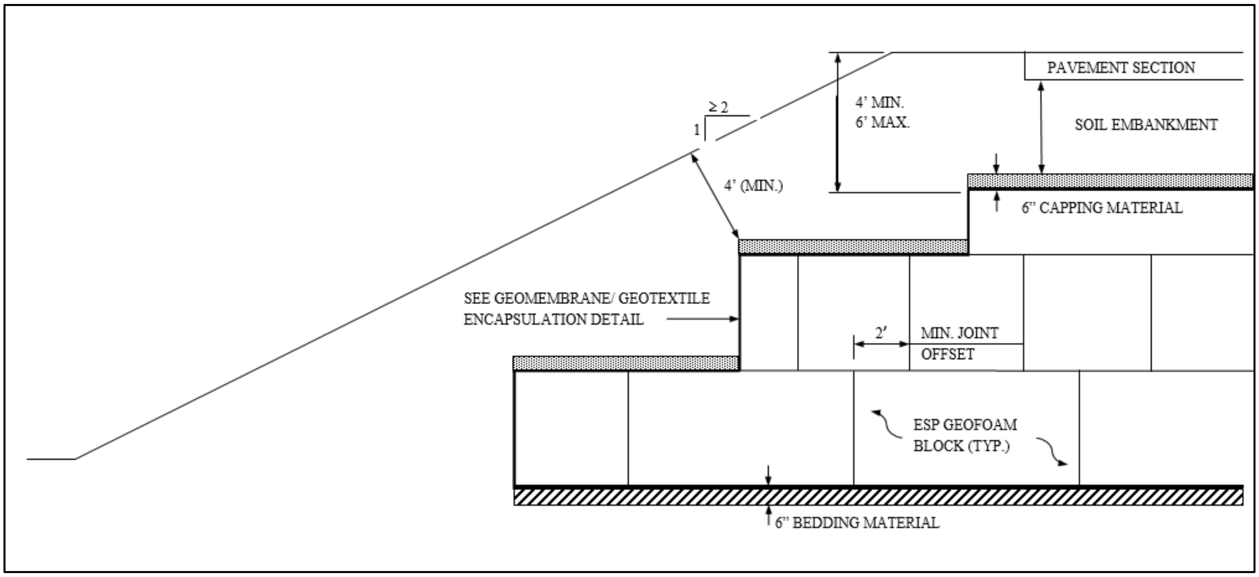


Figure 13.7-3 – Side Slope Detail

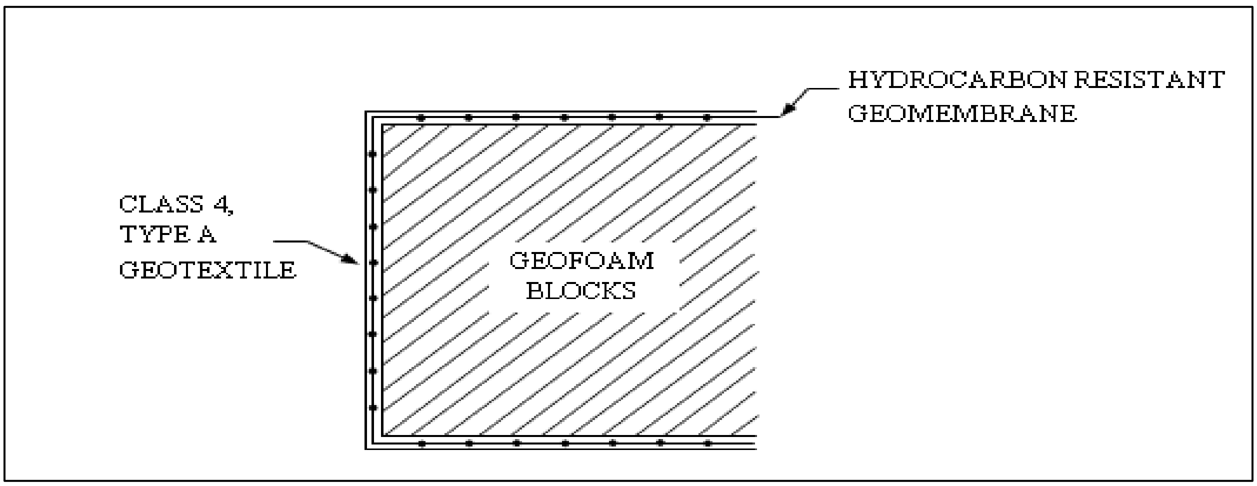


Figure 13.7-4 – Geomembrane Encapsulation Detail

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 most of the geofoam embankment except for 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 required for most 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 number 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, if 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 ±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 ¾ 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 DETERMINATON 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 ($\mathbf{L_{max}}$) = 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:

$$\frac{L_{max}}{No. sets\ of\ dual\ tires} = \frac{21{,}400\ lbs}{2\ sets\ of\ dual\ tires} = \frac{10{,}700\ lbs}{set\ of\ dual\ tires}$$

To determine dual tire contact area (**A$_t$**), use the following equation:

$$A_t = \frac{L_{max}/set\ of\ dual\ tires}{P_t} = \frac{10{,}700\ lbs}{105} = 102\ in^2$$

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:

$$L = \frac{A_t}{W} = \frac{102\ in^2}{20\ in} = 5\ in$$

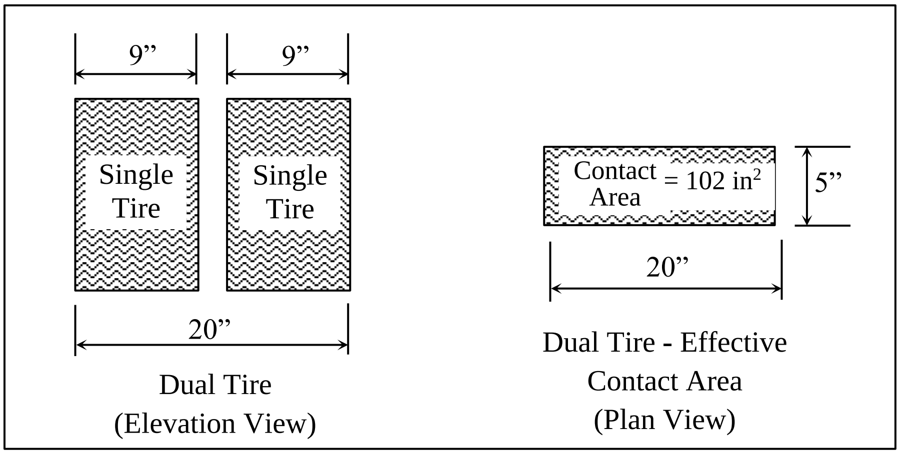


Figure 13.8.1-1 – Dual Tire Elevation View and Plan View

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).

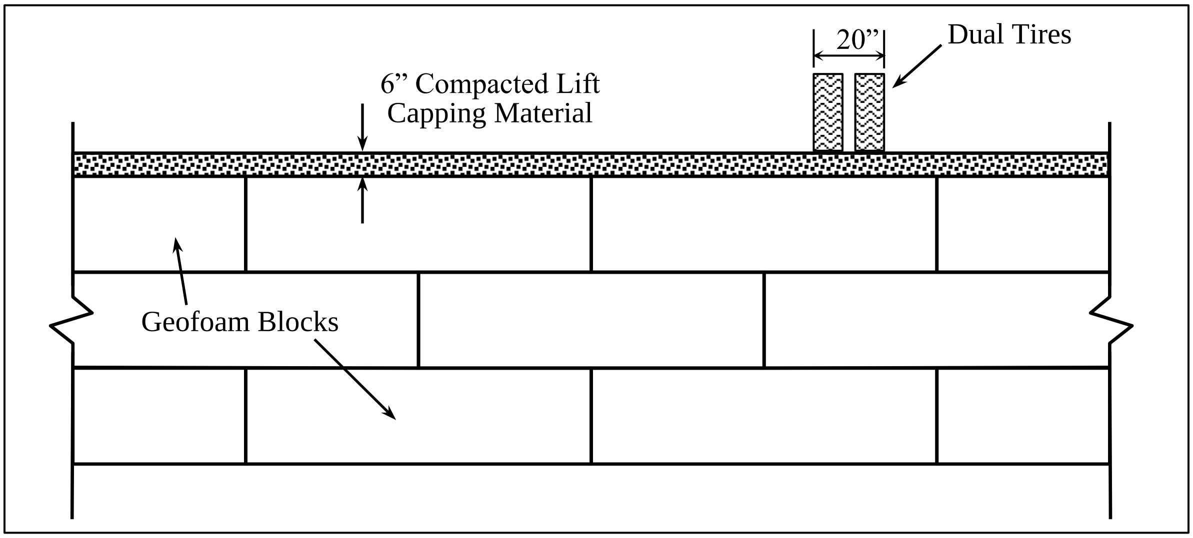


Figure 13.8.1-2 – Typical Geofoam Embankment Section after Placement of 6 Inch Capping Material

- 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).

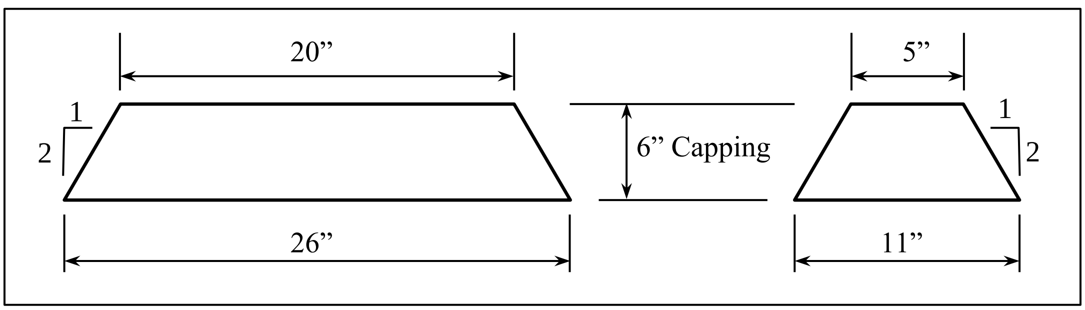


Figure 13.8.1-3 – Pressure Distribution Dimensions on Capping Material

- Determine the loaded area (**A**) at top of geofoam blocks (i.e., bottom of 6 inches of capping material) using the following equation:

$$A = (L)(W) = 11\ in \times 26\ in = 286\ in^2$$

- Determine the equivalent uniform pressure (**P**) at top of geofoam blocks using the following equation:

$$P = (Distributed\ Pressure\ from\ Tires) + (Pressure\ from\ 6\ inches\ Capping)$$

$$P = \frac{L_{max}}{A} + (D_c)(\gamma) = \frac{10{,}700\ lbs}{286\ in^2} + \frac{(0.5\ ft)(130\ {}^{lb}/_{ft^3})}{144\,\frac{psf}{psi}} = 37.9\ psi \sim 38\ psi$$

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 Properties[1]

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).

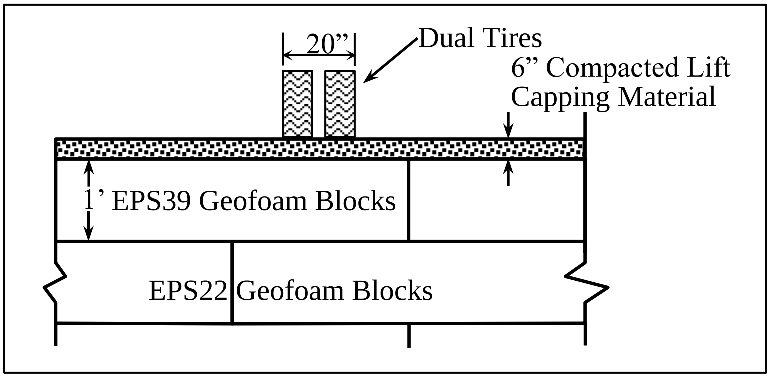


Figure 13.8.1-4 – Typical Geofoam Embankment with 1 foot EPS39 Geofoam

- 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:

$$A = (L)(W) = 23\ in \times 38\ in = 874\ in^2$$

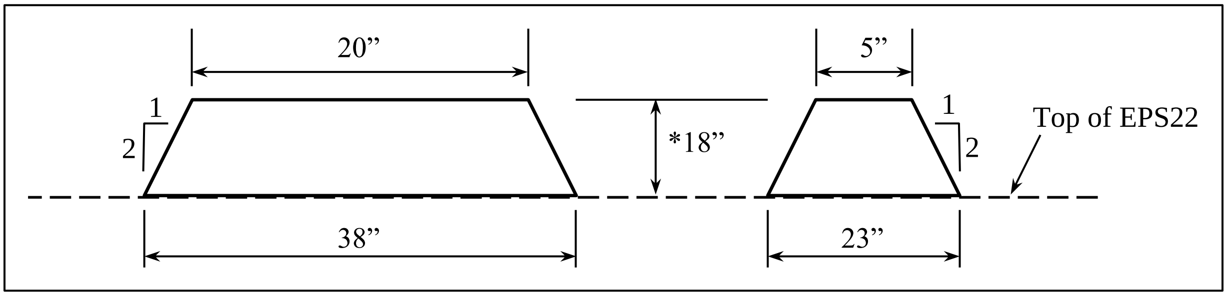


Figure 13.8.1-5 – Pressure Distribution Dimensions of EPS39 Geofoam and Capping Material

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:

$$P = (Distributed\ Pressure\ from\ Tires) + (Pressure\ from\ 6\ inches\ Capping)$$

$$P = \frac{L_{max}}{A} + (D_c)(\gamma) = \frac{10{,}700\ lbs}{874\ in^2} + \frac{(0.5\ ft)(130\ {}^{lb}/_{ft^3})}{144\ \frac{psf}{psi}} = 12.7\ psi \sim 13\ psi$$

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.

$$L_{max} = 10{,}700\ ^{lbs}/_{set\ of\ dual\ tires} \quad \text{(Unless noted otherwise)}$$

To calculate load factor (**IM**) in accordance with DM-4, Part B, 2015, Section 3.6.2.2 using the following equation:

$$IM = 40(1 - 0.125D_E) \geq 0\%, where$$

IM = Dynamic Load Allowance
D_E = Minimum Depth of Cover

For D_E = 4 ft. (i.e., min. required per Publication 408, Section 219)

$$IM = 40\big(1 - 0.125(4\ ft)\big) = 20\% \ \geq 0\% \ \therefore \ \text{OK}$$

Then, using the following equation, determine the total load **(L'_{max})** when applying load factor (**IM**) to the wheel load:

$$L'_{max} = (1.20)\left(10{,}700\ ^{lbs}/_{set\ of\ dual\ tires}\right) = 12{,}840\ ^{lbs}/_{set\ of\ dual\ tires}$$

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.

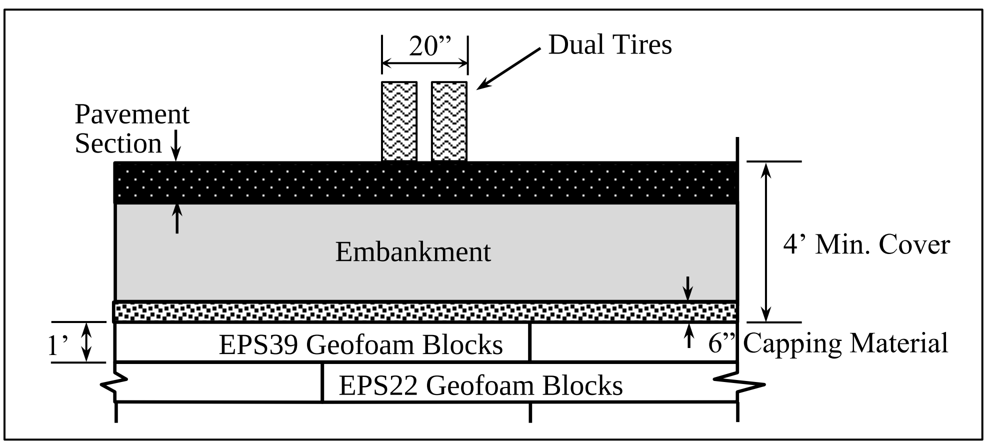


Figure 13.8.2-1 – Typical Geofoam Embankment with 4 feet of Cover

- The Distributed area (**A**) at the top of EPS39 geofoam is determined using the following equation:

$$A_1 = L + 2\left(\frac{D_{min}}{2}\right) = 5\ in + 2\left(\frac{48\ in}{2}\right) = 53\ in$$

$$A_2 = W + 2\left(\frac{D_{min}}{2}\right) = 20\ in + 2\left(\frac{48\ in}{2}\right) = 68\ in$$

$$A = (A_1)(A_2) = 53\ in \times 68\ in = 3{,}604\ in^2$$

- 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.

$$P = \frac{L'_{max}}{A} + (D_E)(\gamma)$$

$$= \frac{12{,}840\ {}^{lbs}/_{set\ of\ dual\ tires}}{3{,}604\ in^2} + \frac{(4\ ft)\left(135\ {}^{lb}/_{ft^3}\right)}{144\frac{psf}{psi}} = 7.4\ psi < 15.0\ psi\ \therefore\ \text{OK}$$

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.

$$P = \frac{L'_{max}}{(68\ in\ + 12\ in)(53\ in\ + 12\ in)} + (D_E)(\gamma)$$

$$= \frac{12{,}840\ {}^{lbs}/_{set\ of\ dual\ tires}}{5{,}200\ in^2} + 3.8\ psi = 6.3\ psi < 7.3\ psi\ \therefore\ \text{OK}$$

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):

For IM = 1.10, $L'_{max} = 11{,}770\ {}^{lbs}/_{set\ of\ dual\ tires}$

For 6 feet, $A_t = 7{,}084\ in^2$

$$P = \frac{L'_{max}}{A_t} + (D_E)(\gamma)$$

$$= \frac{11{,}770\ {}^{lbs}/_{set\ of\ dual\ tires}}{7{,}084\ in^2} + \frac{(6\ ft)(135\ {}^{lb}/_{ft^3})}{144\frac{psf}{psi}} = 7.3\ psi < 15.0\ psi\ \therefore\ \text{OK}$$

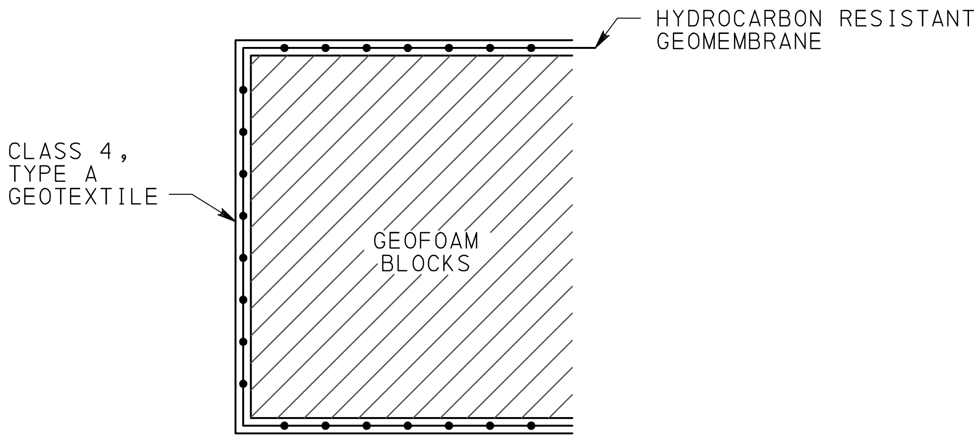
- For 1 foot below top of EPS39 geofoam (i.e., at top of EPS22 geofoam) for 6 feet of cover. Assume the dead load is relatively unchanged from 1 foot of EPS39 geofoam. Add additional pressure distribution to account for 1 foot of EPS39 geofoam. Using the same factored load above:

$$P = \frac{L'_{max}}{(92\ in\ + 12\ in)(77\ in\ + 12\ in)} + (D_E)(\gamma)$$

$$= \frac{11{,}770\ {}^{lbs}/_{set\ of\ dual\ tires}}{9{,}256\ in^2} + 5.6\ psi = 6.9\ psi < 7.3\ psi\ \therefore\ \text{OK}$$

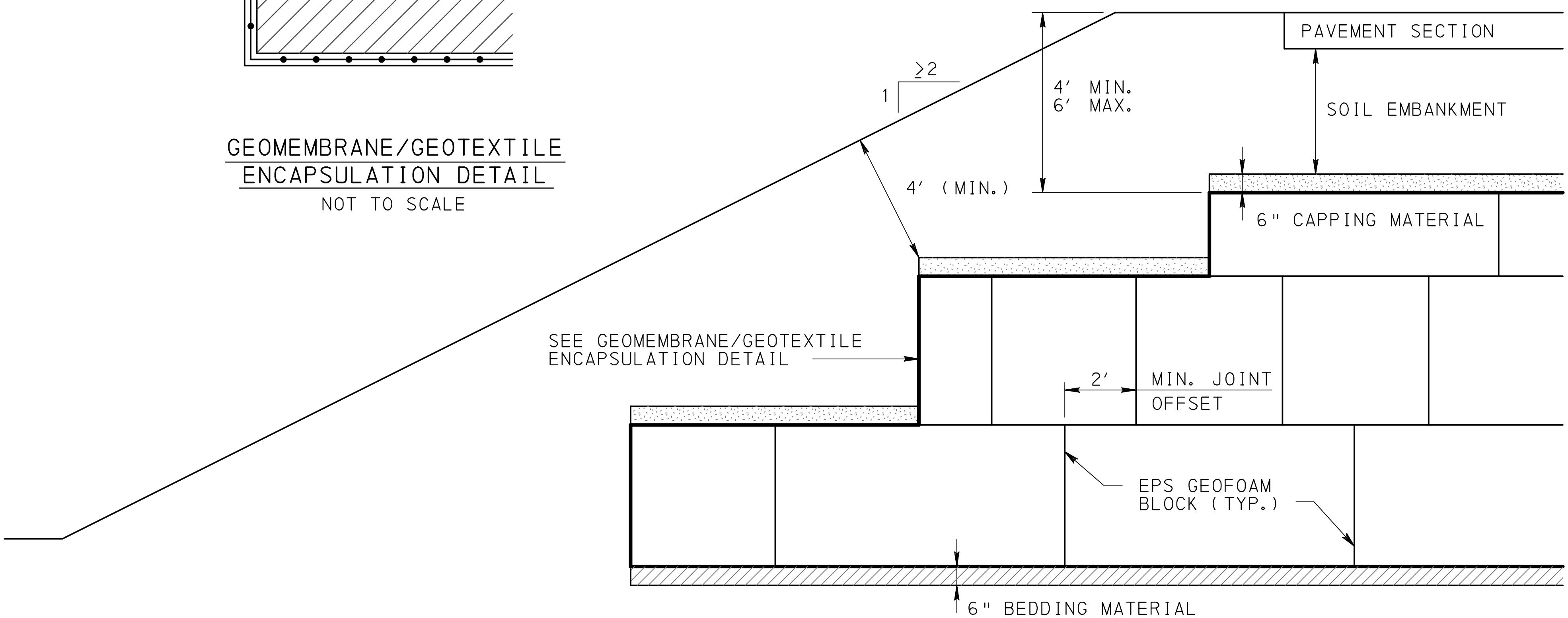
In conclusion, the estimated pressures for the final loading condition were calculated for cover (i.e., pavement section, embankment and capping material) thickness at 6 feet. Based on the calculations, the pressures at the top of the geofoam block fill (i.e., EPS39 geofoam) and 1 foot below the top (i.e., EPS22 geofoam) are below the 1% compressive resistance of the geofoam.

FILE: C:\PENNDOT DETAILS\GEOTECH DETAILS\Geofoam Detail.dgn
DATE: 27-Feb-2014 10:01



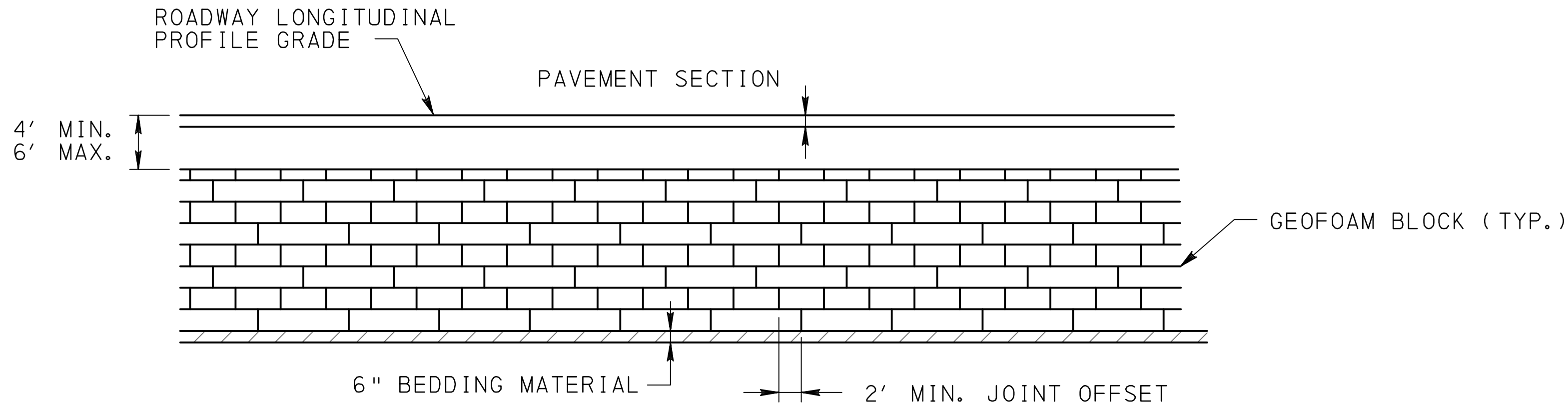
GEOMEMBRANE/GEOTEXTILE ENCAPSULATION DETAIL

NOT TO SCALE



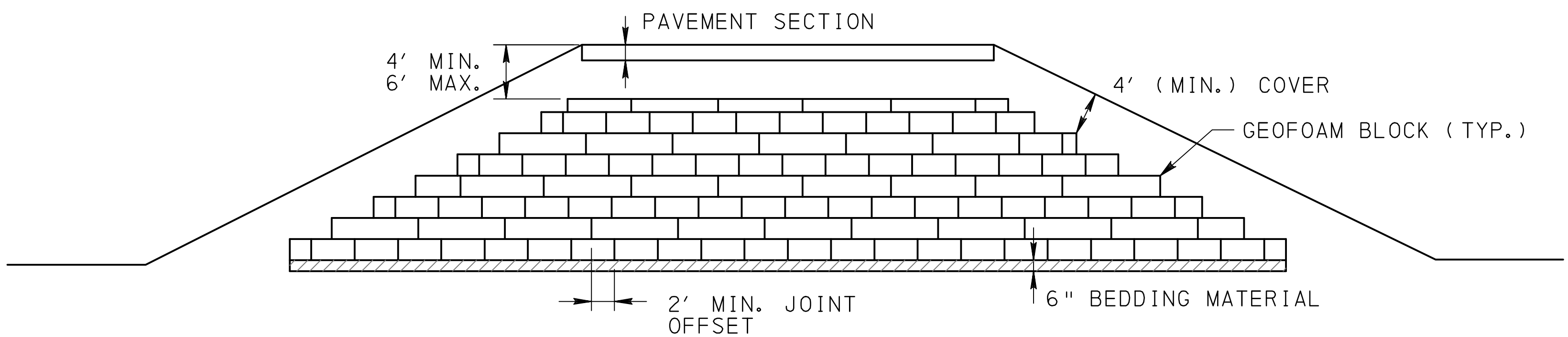
GEOFOAM SIDE SLOPE DETAIL

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

RESERVED FOR
FUTURE DEVELOPMENT OF:

PUBLICATION 293
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 14 – GEOSYNTHETICS USE GUIDELINES

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
GEOTECHNICAL ENGINEERING MANUAL

CHAPTER 15 – ROCK SLAKE DURABILITY

CHAPTER 15 – TABLE OF CONTENTS

15.1 INTRODUCTION

This chapter of this publication 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 that 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 that 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.

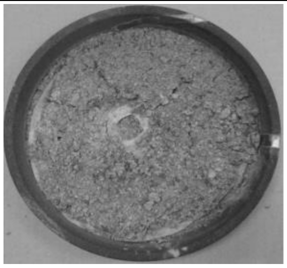
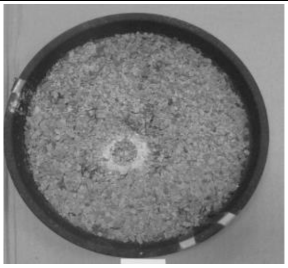
6. **<u>Slaking</u>** – The disintegration of rock under wetting and drying conditions and when exposed to air. This behavior is related primarily to the chemical composition of the rock.

15.1.3 Overview of Slaking

Fine-grained sedimentary rock that 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 Samples[1]

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 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.

Most 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 that 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 that 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 Formation[1]	Slaking Potential[2]
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.

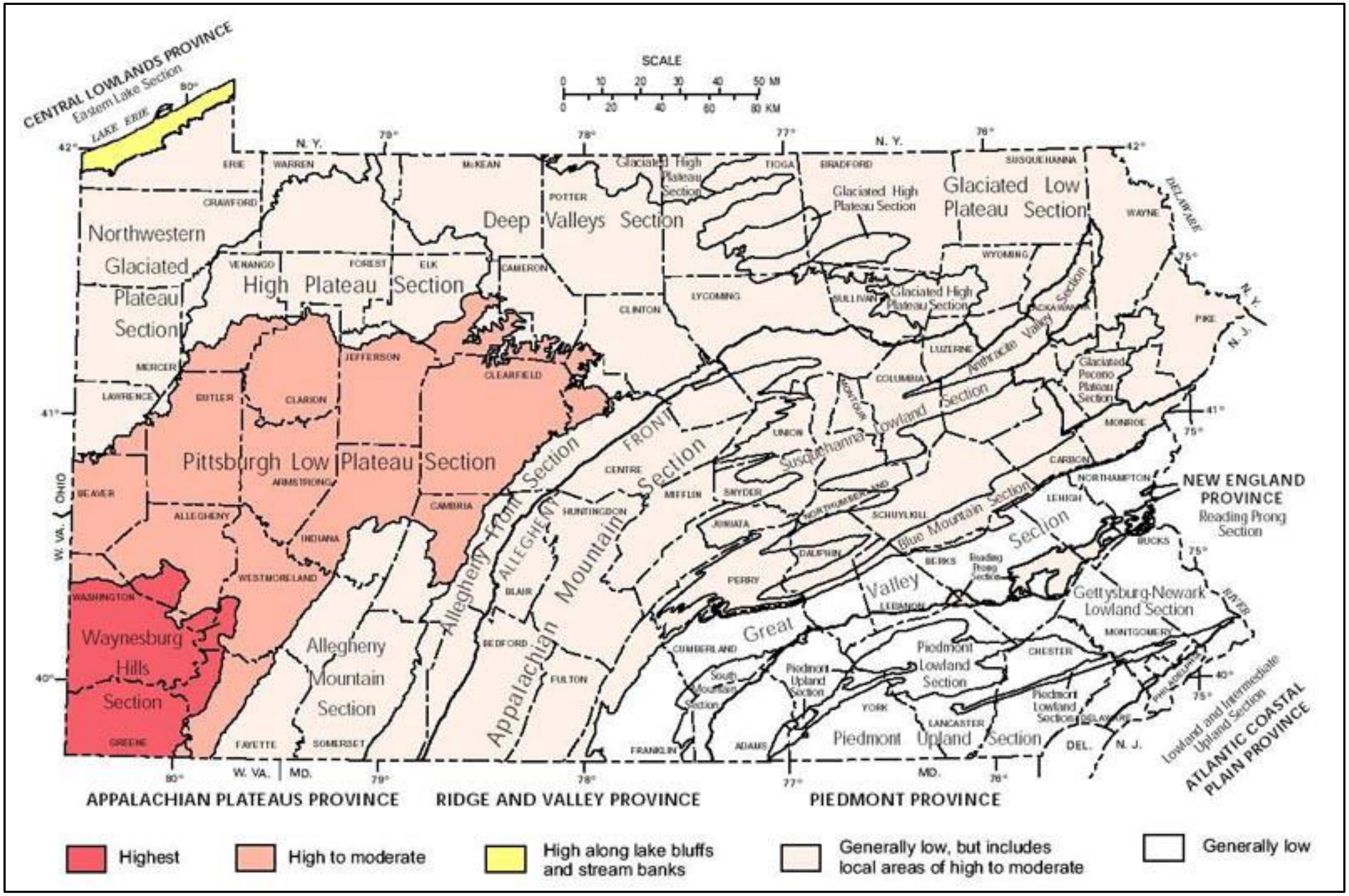


Figure 15.3.2-1 – Landslide Susceptibility Map of PA (PA-DCNR)

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 that 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 discretional, but recommended, for the foundation design. These values may assist in determining the extent of slake testing needed and 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 is 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 (i.e., 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. According to 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.

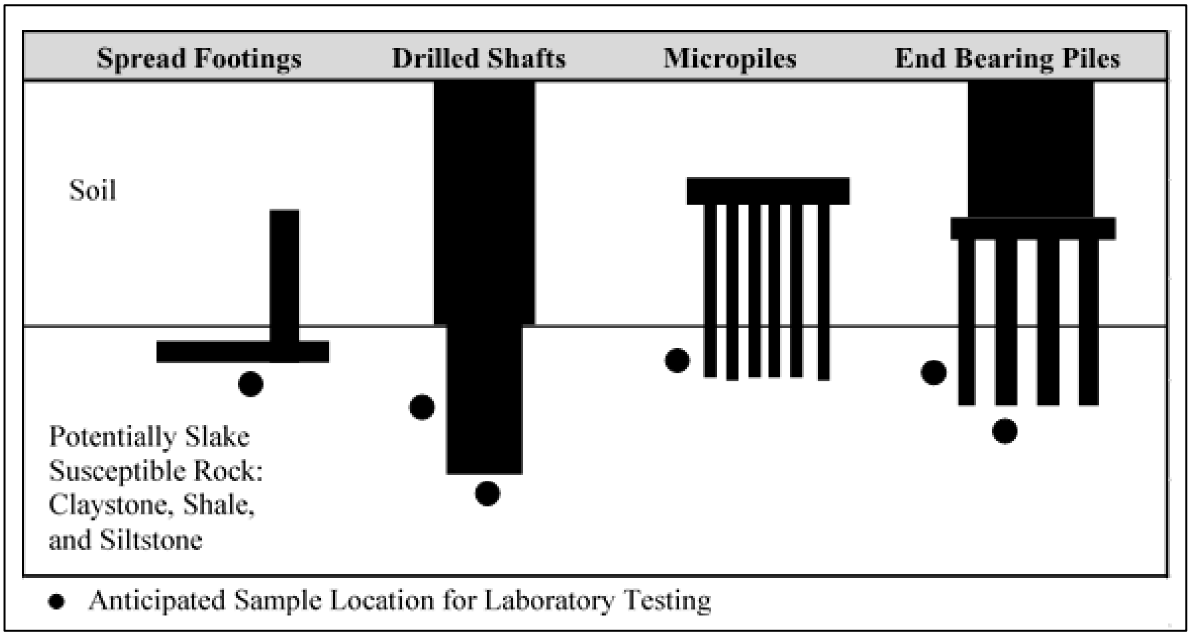


Figure 15.3.4-1- Guidelines for Laboratory Sample Location Selection

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

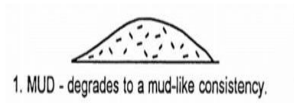
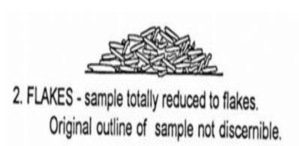
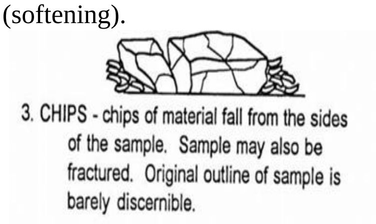
The Jar Slake Test is a relatively simple, inexpensive test that is intended to quickly assess the overall slake potential of rock. This test can conclusively identify rocks that are significantly prone to slaking (i.e., low slake durability). This test is also used to determine if more detailed testing, 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.

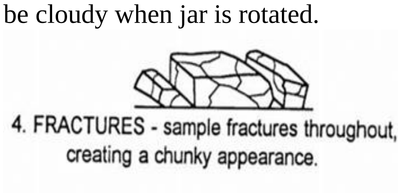
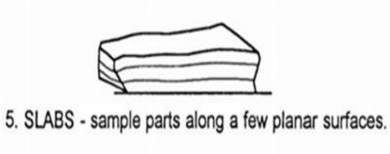
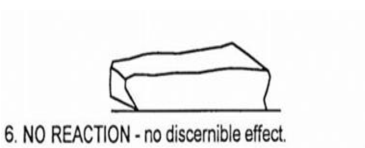
The Jar Slake Test method that is to be used on Department projects is detailed in PTM No. 122. This PTM closely follows the jar slake test method developed by Deo (Use of Shale in

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 according to ASTM D4644) is required.

Table 15.4.1-1- Jar Slake Test Interpretation Guideline

<table>
<tr><th>Observations of Rock Sample During Jar Slake Test</th><th>Example Photograph</th><th>I_J</th><th>Material Interpretations</th><th>Further Slake Durability Testing Required?</th></tr>
<tr><td>Rapid breakdown (several minutes or hours) of entire sample into fine flakes and/or fine sediment (mud).

</td><td rowspan="2"></td><td>1</td><td rowspan="2">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.</td><td rowspan="2">No.</td></tr>
<tr><td>Breakdown of majority of sample into fine sediment and flakes.

</td><td>2</td></tr>
<tr><td>Breakdown of sample into separate rock fragments (fracturing) with significant surface slaking (softening).

</td><td></td><td>3</td><td>Same as material interpretation for I_J values of 1 and 2.</td><td>No.</td></tr>
</table>

Observations of Rock Sample During Jar Slake Test	Example Photograph	I_J	Material Interpretations	Further Slake Durability Testing Required?
Breakdown into rock fragments (fracturing) with minor surface slaking (softening). Soak water will be cloudy when jar is rotated.  		**4**	**Likely Slaking Material:** Rock is likely slake prone. May not slake under short-term exposure. May require mitigation if used to provide foundation resistance.	Yes. Proceed to 15.4.2.
Very minor fracturing of sample is evident with no surface slaking (softening) after 24-hour soak. Soak water remains clear or nearly clear.  		**5**	**Possibly Slaking Material:** Rock is possibly slake prone. May require mitigation if used to provide foundation resistance.	Yes. Proceed to 15.4.2.
No change to condition of the rock sample discernable after 24-hour soak. Soak water remains clear.  		**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., IJ value of 4 to 6), more rigorous testing using the Slake Durability Test method per ASTM D4644 is required. Like 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).

$$I_{D(2)} = \left(\frac{Dry\ Weight\ Retained\ (after\ two\ cycles)}{Dry\ Weight\ (before\ test)}\right) x\ 100\%$$

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 that 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" 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.

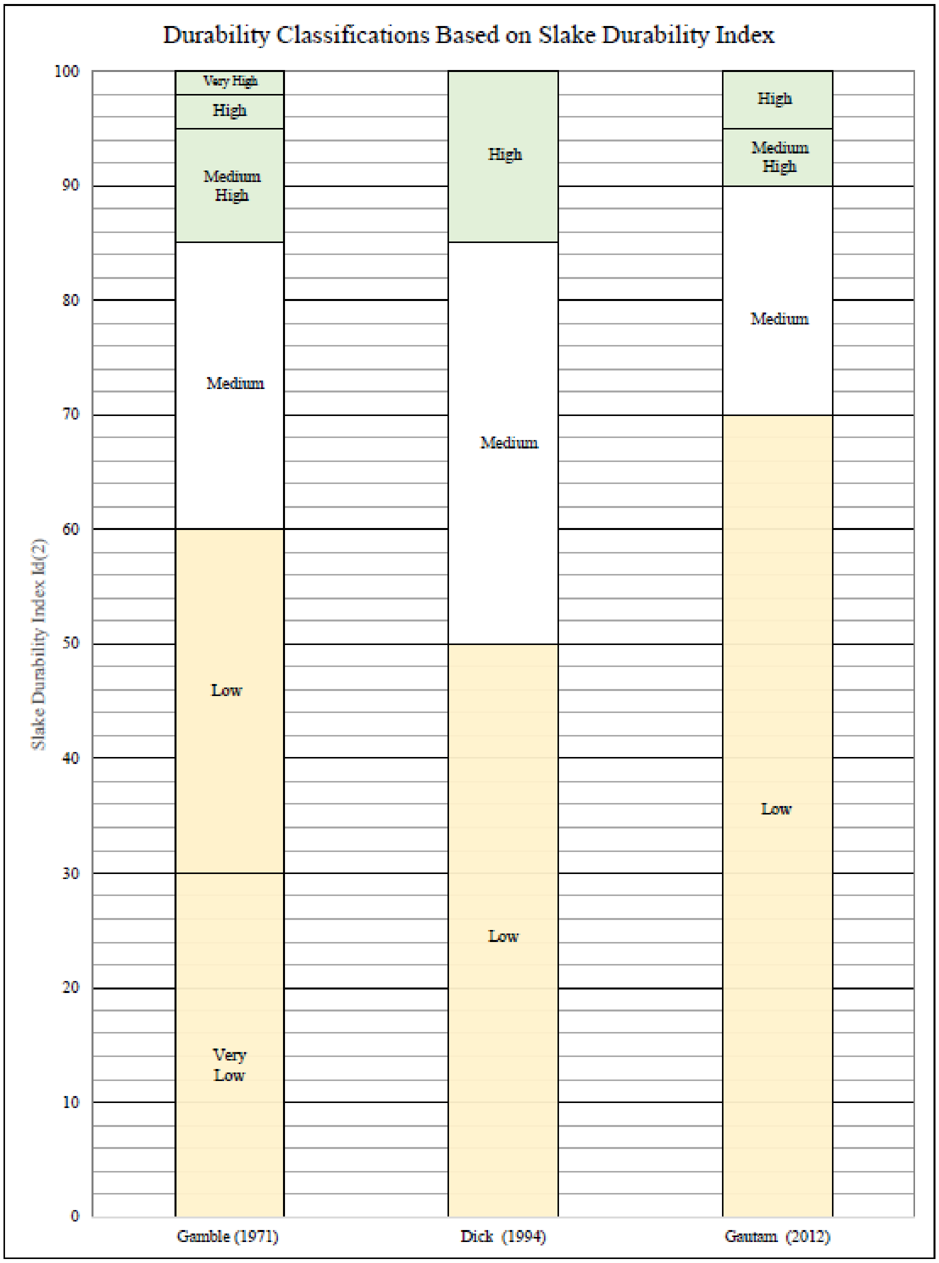


Figure 15.4.2-1- Comparison of Published Slake Durability Classifications

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[1]	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%

Notes: 1. Material interpretation descriptions developed from a review of various sources and studies including ASTM D4644. Material interpretation descriptions are consistent with Type I, II and III pictorial representations in ASTM D4644.

These general interpretations are anticipated to be applicable to most 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 experience when designing a foundation in slake-prone rock. District Geotechnical Engineers (DGE's) are encouraged to consult with the Chief Geotechnical Engineer (CGE) 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, 2nd 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, 2nd 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 (e.g., sound barriers, high mast lights, certain ITS equipment, etc.)
> 85%	No consideration for slake is needed for design or construction on high-durability rock.		

RESERVED FOR
FUTURE DEVELOPMENT OF:

PUBLICATION 293
GEOTECHNICAL ENGINEERING MANUAL

APPENDIX A – INSTRUMENTATION FIELD (IN-SITU) TESTING

RESERVED FOR
FUTURE DEVELOPMENT OF:

PUBLICATION 293
GEOTECHNICAL ENGINEERING MANUAL

APPENDIX B – CLAY AND IMPERVIOUS LINERS

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PUBLICATION 293 –2022
GEOTECHNICAL ENGINEERING MANUAL

APPENDIX C – LIMITED MOBILITY GROUTING USE GUIDELINES

APPENDIX C – TABLE OF CONTENTS

C. 1 INTRODUCTION

This appendix of this publication provides guidance for the use of Limited Mobility Grouting (LMG) for void filling, sinkhole mitigation, and soil densification applications on transportation related projects. LMG, for the purpose of this chapter, is defined as low slump grout such as compaction-type grout, that does not travel freely and that becomes immobile when injection pressure ceases. Compaction grout is a subset of the broader limited mobility grout umbrella and is defined below.

The desired LMG material is a very low-slump soil-cement mix grout (typically one to two inch slump, or less). This limits the size of rock fractures that the grout can penetrate and the distance the grout can migrate in smaller solution features. Solution features in carbonate bedrock are often highly interconnected, over long distances; therefore, this helps prevent excessive grout takes that could occur with more fluid grouts. The viscous nature of the grout also limits lateral migration of the soil during injection in overburden materials allowing for controlled placement of the grout. Grout injections are typically made using a bottom-up approach with injection stages of one or two feet.

Compaction grout is a specific type of LMG with special properties and is typically batched and mixed on-site, whereas, LMG for void filling, is typically batched at a concrete plant and delivered to the site in a ready-mix truck. Compaction grout, within the context of this chapter, is grout injected with less than one-inch slump. It normally consists of a soil-cement with enough silt sizes (to provide mobility) mixed with sufficient sand and gravel sizes (to develop sufficient internal friction) to cause the grout to act as a growing mass as injection continues under pressure. The grout generally does not enter soil pores (except where open-work boulder gravels are present), but remains a homogeneous mass that gives controlled displacement to compact loose non-plastic soils, for lifting structures, or both.

Compaction grouting consists of injecting a stiff mortar-like grout into the soil. This mix will not fill voids, but rather expands as a globular mass under pressure that displaces and densifies the surrounding soil. Since compaction grouting's intended use is densification of soil, its' application is limited to treatment of the overburden, and not rock. Compaction grouting can be applied to soils above and below the groundwater table. Compaction grouting is most effective in soils that can be densified by squeezing water and air from void spaces. Cohesive soils, such as soft clays are not well suited for compaction grouting due to the build-up of excessive pore pressures and potential for hydraulic fracturing. In this chapter, the term Limited Mobility Grouting (LMG) for soil densification is generally used for compaction grouting.

Please note that this chapter is only intended to guide the Engineer in the design of an LMG program. Experience and engineering judgement must also be used on a project-by-project basis per site-specific subsurface conditions. This guide is not intended for use with high mobility or fluid grouting applications (e.g., filling mine voids, abandoned conduits, etc.).

C.1.1 Advantages of Limited Mobility Grouting

The use of LMG allows a precise area of treatment with little to no waste-spoil disposal. The use of site batching allows for adjustments to grout mixes to be made at the site. On-going review of grouting placement data allows for adjustments to the design to be made based on site specific conditions. Large equipment is not required to drill grout holes and inject grout. Grout holes can be drilled vertically or at an angle in order to treat confined areas, zones difficult to access, and areas below structural foundation members.

C.1.2 Disadvantages of Limited Mobility Grouting

The use of LMG for soil densification requires confining overburden pressures typically five feet or more of overburden depth to react against the grouting pressures and is not appropriate for cohesive soils. LMG operations for void filling and sinkhole mitigation do not have these limitations. Effective design of LMG requires a body of successful work and experience. A standardized design methodology (national design specification) is not available.

C.1.3 Effective Limited Mobility Grouting Program

An effective LMG Program requires implementation of the following elements:

- Experienced contractors and construction inspectors
- A thorough monitoring program during grouting with regular collection and review of data
- Regular, timely submission of accurate, legible reports to the Department documenting grout placement pressures, flow rates, and volumes by stage
- Timely review of results, decision making, and communication of adjustments to the program including grout hole pattern/location, grout mix, placement pressures, and allowable flow rates
- A thorough QA/QC testing plan that is followed during operations
- A robust verification testing plan.

The Contractor is solely responsible for the collection, quality, accuracy, maintenance, and timely distribution of all LMG records to all contributing parties involved in the decision-making process.

C.1.4 Understanding LMG and Soil Densification

LMG is typically injected from the bottom up (upstage), creating a continuous grout column. LMG is a strain-controlled process. The pumping rate must be controlled so that excess pore pressures are not developed. Note that slow low mobility grout injection allows a greater volume of injection, as rates greater than two cubic feet per minute (cfm) will likely cause hydraulic fracturing and loss of injection control. For sensitive applications, where hydraulic fracturing of weak soils, ground heave, or movement of adjacent structures is of critical concern, injection rates as low as one-half cfm may be required. Regardless of the slump or stiffness of the grout, if it is unduly mobile or behaves as a fluid in the ground, hydraulic fracturing of the soil can occur, resulting in loss of control.

LMG is less effective near the ground surface due to the limited overburden pressure that provides little surface restraint for densification of soils, thus results in ground heave. If densification of shallow soil is a requirement, top down (downstage) injection provides additional restraint and containment for lower stages. The grouting sequence should be carefully planned to maximize the benefits of the grouting process. See Section C.4.8 for additional details on grout sequencing.

During LMG (for soil densification), the grout/soil interface exceeds the yield stress of the soil and causes permanent densification to occur. As the grout is pumped into the existing soils, the soil is densified and deformed in a limited zone around the injected grout. The soil reacts elastically beyond the zone of plastic deformation. The size of the elastic and plastic deformed areas depends on the amount of grout injected and the soil characteristics. The staged grout column construction acts to prevent vertical grout expansion. For sands and silty sands, lateral stress is typically lower than the vertical stress. The direction of least resistance for grout flow is the lateral direction.

Soils most suitable for LMG (for soil densification) have the ability to densify without hydraulically fracturing, without having grout permeate into the pores of the soils (as could occur with gravel), and to drain during grouting.

During grouting, the soil is subject to compression while also being sheared, causing a change in the volume. The shear component of volume change is generally much larger than that caused by increasing the mean stress. Loose soils suitable for soil densification through LMG become denser when sheared. Sands with descriptions of dense to very dense will likely dilate rather than densify during compaction grouting. Likewise, the requirement for partial drainage generally excludes normally consolidated, high plasticity clay soils from being densified using LMG. LMG for soil densification is most effective for mixed granular soils such as sandy gravels, silty sands, clayey sands, silts, and some low plasticity clayey soils when soils are loose to medium dense. LMG soil densification is not generally effective for fine-grained soils such as high plasticity clays, and clays below the water table.

The quality of proposed LMG for soil densification is affected by the homogeneity of the soil, cohesive and frictional strength of the soil, compressibility of the soil, soil permeability, and presence and distance to subsurface structures, subsurface openings or slopes. Furthermore, it is critical to have a thorough understanding of the subsurface conditions to determine suitability and potential effectiveness of a LMG program. Section C.3 provides a discussion of subsurface investigation elements that are often used in developing a LMG program.

C.2 LIMITED MOBILITY GROUTING APPLICATIONS

The objective of a limited mobility grouting program may include one or more of the following:

1. Karst Mitigation

    a. Fill voids in karst bedrock into which overburden soils can migrate.
    b. Choke off the openings, or sinkhole "throats", in the top of the rock mass where soil migrates into the rock.
    c. Fill voids or densify loose and soft zones in the overburden soils caused by previous sinkhole activity.

2. Foundation System Improvement

    a. Densify loose existing fill or improperly compacted fill below existing or future structures.
    b. Densify loose in-situ soils below existing or proposed structures.
    c. Fill voids in karst bedrock before pile driving to top of rock, also known as cap grouting.
    d. Fill voids along the perimeter of drilled shaft casing to establish contact between the casing and soil providing adequate lateral load support.

3. Other Applications

    a. Structural slab jacking, pavement jacking, etc.
    b. Improve liquefiable soils.

The most common LMG applications on Department projects are soil densification below proposed structures, densification of improperly compacted fill below existing or proposed structures, and void filling in karst for spread footing foundations.

C.3 SUBSURFACE EXPLORATIONS FOR GROUTING

The primary purpose of a subsurface exploration for LMG is to obtain enough information to determine if LMG is a suitable alternative, and to design a successful grouting program. Project sites often encounter variability in subsurface soil and rock conditions, which in turn, impact the effectiveness of the employed grouting methods. Aspects related to planning of a subsurface exploration program are discussed below. A detailed discussion and resources for office and field reconnaissance are provided in Chapter 2 of this publication.

C.3.1 Office Reconnaissance

Available geotechnical and geologic data should be obtained and reviewed. Where available or applicable, obtain previous geotechnical and environmental studies or reports (e.g., Phase I Environmental Site Assessment (ESA), etc.) from the project area or adjacent properties. Additionally, the following information/sources should be reviewed:

- Topographic maps – recent and historical published and site-specific project mapping
- Aerial photography

- USDA Soil Survey
- Bedrock and surface geology mapping - geologic maps should be reviewed to get information relating to the regional, local, and site geology of an area.
- Sinkhole mapping and Open File Reports
- Mine maps
- Utility mapping and documentation, including drainage pipe inspection videos
- Well records from the Pennsylvania Groundwater Information System (PaGWIS) or other sources

C.3.2 Field Reconnaissance

Field reconnaissance should be the next step in planning a grouting program, as it allows for a general characterization of the area and consequently provides a starting point to plan a site-specific exploration program to address the issues identified. In general, the following factors should be evaluated during the field reconnaissance:

- Geologic site characteristics and potential geologic hazards
- Surface drainage features and waterways
- Subsurface drainage structures
- Proximity to existing structures or surface facilities (on and off the right of way)
- Existing utilities (overhead and underground) and potential conflicts
- Access limitations for drill rigs, geophysical equipment, etc. during subsurface investigation
- Access limitations for drilling and grouting equipment during grouting program
- Site features that could impact geophysical exploration methods
- Traffic control considerations
- Potential need for permits (e.g., work near or access through streams, etc.)
- Potential impacts to adjacent property owners

During the field reconnaissance, photographs and videos should be taken of the site features, and all relevant observations should be documented using photo logs, site reconnaissance plans, diagrams, and more. Field measurements of specific features should be taken as necessary to supplement existing mapping or provide additional information for planning of the grouting program.

Where applicable, attempts should be made to interview residents, business owners, and Department personnel (engineering and maintenance employees) regarding the history of observed surface features and roadway performance within the project area. Additionally, for karst-related projects, interviewees should be asked for information regarding the history of development of surface features and repairs in the project area.

C.3.3 Geophysical Survey

Geophysics can be an important site characterization tool and can be used to supplement test boring information at a site, allowing for an evaluation of the physical properties of soil and rock between and beyond the limits of test boring locations. However, geophysical methods should not be used as a replacement for conducting test borings. The appropriate geophysical method(s) should always be carefully selected based on the site geology, local terrain, potential obstructions, accuracy, and limitations of the various geophysical survey methods, magnitude of the anticipated hazards, objectives of the survey, necessary traffic control methods including consideration of traffic impacts (vibration), working hours, and cost of the project.

A geophysical survey should be considered in advance of the test boring investigation to provide a preliminary assessment of the subsurface conditions at the site, and to aid in planning the test boring program when karst conditions/voids, soft soils, or pinnacle conditions are anticipated. Properly selected geophysical methods can be used to identify the location of anomalous subsurface conditions (e.g., voids, soft or loose soil zones, etc.). Additionally, geophysics can be used to profile top of rock, identify the locations of certain types of utilities, and identify seepage patterns.

Geophysical data may be useful in the planning of a LMG program, by aiding in the determination of the area to be grouted, while targeting identified anomalies of interest. In addition, geophysical profiling methods can be used to aid in estimating the approximate grouting treatment depths that are required. This allows for a better prediction of the costs associated with the grouting program, particularly the drilling costs.

Common geophysical profiling methods used in conjunction with planning of LMG programs include Multi-channel Analysis of Surface Waves (MASW) and Electrical Resistivity Imaging (ERI). Ground Penetrating Radar (GPR) can be used for near-surface profiling, such as investigating potential voids beneath the pavement section, or utility locations. Microgravity mapping can be used to provide planimetric site coverage. Typically, more than one type of geophysical method is used at a site to provide complimentary data sets.

C.3.4 Test Boring Exploration Program

A sufficient number of test borings should be performed to obtain an accurate depiction of the subsurface conditions, focusing primarily on determination of the extent and variability of the stratigraphy encountered at the project site. In structure areas, reference Chapter 3, Table 3.2.4-1 as a starting point to determine the number of borings. In roadway areas, reduce boring spacings from those noted in Table 3.2.4-1 to 100 feet or less, depending on the size of the area and the variability of the subsurface conditions. For small areas, a minimum of two borings should be performed. Keep in mind that depending on the form of verification testing used, the results of the exploratory boring program may be a basis for comparison with post grouting conditions.

If a site geophysical survey is performed as part of the subsurface exploration, the data can be used in planning the test boring program. Areas of interest from the geophysical survey, particularly identified anomalies, can be targeted based on specific locations and depths in the test borings. It is also important to select boring locations that are not in areas of anomalies, to help correlate the geophysical survey results with the test boring information.

Where geophysical survey data is not available, existing information should be carefully reviewed, and test borings should extend to a depth that allows characterization of competent bearing material, at a minimum. Grout will add weight to the treated formation; therefore, it is important to be able to verify that the bearing material can support that additional load without causing detrimental settlements at the ground surface.

During drilling of the test borings, it is important to note the following, among others:

- Changes in the drilling penetration rates
- Depth to ground water
- Changes in groundwater elevation
- Presence and extent of organic matter
- Presence, extent and composition of soils and fill
- Depth, type, and condition of bedrock

Any drilling obstructions and anomalies encountered should be described in detail, and where rocks are encountered, the size and hardness of the material should be indicated including the method used to penetrate or displace the rock. All borings must extend into competent materials capable of supporting the overlying proposed structure, soils, and proposed grout. Provide additional borings outside of the structure footprint as needed to properly characterize zones or areas that may also adversely influence or be affected by the proposed or existing structure(s).

In karst areas, a minimum of 20 feet of rock coring below top of rock is recommended, but if ten feet of continuous competent rock is encountered, the hole may be terminated. HQ-size core barrels or spin casing that can be advanced into rock are recommended to facilitate standard two-inch O.D. SPT sampling where soil-filled features are encountered below apparent top of rock.

C.3.5 Laboratory Testing

Laboratory testing is recommended for all LMG applications, but it is particularly important for LMG for soil densification. All soil types in the treatment zone should be characterized by performing moisture content, gradation with hydrometer, and Atterberg Limits. If LMG for soil densification is being considered for low plasticity fine grained soils, consolidation properties are useful for evaluating the soil response to pore pressure changes. Laboratory testing should also be performed on the soil layer(s) identified as the bearing material to verify that the bearing material can support the additional load provided by the LMG injection without causing detrimental settlement at the ground surface or at structure locations.

Additional guidance for the applicability of laboratory tests can be found in the ASCE Standard, Compaction Grouting Consensus Guide [ASCE/G-I 53-10].

C.4 LIMITED MOBILITY GROUTING PROGRAM DESIGN

For developing a LMG Program, experience-based design is commonly employed; the following sections provide program design guidance. Design of the final LMG program is dependent on information obtained during implementation. A successful LMG program relies on the experience of the contractor and agency, and the quality of the information gathered during the subsurface exploration, construction, and verification testing phases of work. A successful LMG program should involve a continuous, managed, and integrated process of program design, construction control, monitoring and review, and evaluation of grouting results. Appropriate modifications can then be made to program methods and procedures leading to successful incorporation of the program changes into construction.

Unless indicated otherwise, the following discussions are applicable to LMG for the karst mitigation and foundation system improvement applications as presented in Section C.2.

C.4.1 Defining Limits and Depth of Treatment

The limits of the grouting program should be selected based on the results of the subsurface exploration and goal of the program. For void filling, soil densification, and karst mitigation, the planimetric limits of the program should encompass known surface subsidence features, repair areas, and areas of interest identified by the test borings and geophysical data, including soft or loose soils, voids, potential sinkhole throats, low shear wave anomalies, and low microgravity anomalies. For foundation improvement for structures or embankments, the planimetric limits of the program may be controlled by the stress distribution of the loaded area through the proposed treatment zone.

The execution of a LMG program is a dynamic process, and it may be necessary to adjust treatment depths as the program proceeds to address site-specific conditions identified by the Preconstruction Test Grouting Program or ongoing production grouting. Unless top down injection is being performed, no pressure injection of grout should be performed at depths shallower than approximately eight to ten feet below ground surface, due to the likelihood of ground heave. In karst applications, the grout holes should extend a minimum of ten feet below the top of rock, and potentially deeper if warranted by site conditions. It may be necessary to extend the zone of treatment deeper in areas where rock quality is particularly poor or where voids are encountered within the rock mass (either during the subsurface investigation or during the grouting program).

C.4.2 Grout Hole Spacing

Primary grout hole spacing typically ranges from six to twelve feet center-to-center. A reasonable initial spacing for primary grout holes is a nominal eight to ten-foot square pattern. Grout hole spacing may be adjusted where required to facilitate staging of traffic control patterns or to avoid conflicts with existing utilities or existing structures. A square grouting pattern facilitates calculation and consistent location of secondary and tertiary holes. Avoid triangular or other grid formats that are not easily divisible to establish location of secondary and tertiary holes. Such grid patterns can lead to difficulties in determining the location of secondary and tertiary grout holes and appropriate locations for verification testing to ensure soil improvement has occurred between later stages of grouting.

Spacing can be impacted by grout injection rate. A grout injection rate that is too high can result in soil behaving in an undrained manner. On the other hand, a closer grout hole spacing may become necessary if the amount of grout that the soil can accept is decreased.

C.4.3 Grout Mix Design

Details regarding the grout mix requirements are to be provided in the special provision, and the special provision is to require that the Contractor's proposed grout mix be submitted to the Department for review and approval. A standard special provision is available on ECMS. General requirements for the grout mix are provided below.

- Typically, the grout mix includes cement, filler and water, and other approved additives, as needed. Constituents of the grout mix can be varied in order to achieve a pumpable mix with a slump less than two inches. Grout slump should be measured from samples taken at the pump hopper. Granular breaks as the extrusion exits the hole will occur with an acceptable grout mix.
- Aggregate should be rounded (non-angular) to prevent particles from locking together during pumping. The gradation of the aggregate should be such that the particles do not result in blockage of the pumping mechanism.
- Cement is added to provide structural capacity to the injected mass. If structural strength is not required, reduce the compressive strength requirements for the grout.
- Additives can include fly ash, plasticizers, high range water reducers, anti-washout agents, and viscosity modifiers. Fly ash may be used for compaction grouting to facilitate pumping of material. Use chemical admixtures with caution as they can affect internal friction and can affect grout quality.
- Where treatment of open voids is required, use of reduced slump and/or addition of fine gravel to the grout should be considered.
- The 28-day compressive strength of the grout should be selected based on the treatment objectives and may range from 100 psi to 2,000 psi or more. Strengths as low as 100 psi may be appropriate for soil densification. A specified compressive strength of 400 psi is usually appropriate for LMG under paved or open areas. Higher strengths should be considered where structural support is involved. Compressive strengths as high as 3,000 or 4,000 psi may be desirable for cap grouting of rock before driving point bearing piles.
- Additional guidance for the grout mix requirements can be found in the ASCE Standard, Compaction Grouting Consensus Guide [ASCE/G-I 53-10].

C.4.4 Grout Injection Rate

Lower injection rates allow more grout to be placed before hydraulic fracturing or surface heave occurs; therefore, maximize the effectiveness of a LMG program.

- An optimal injection rate should be determined during the Preconstruction Test Grouting Program at the beginning of construction, by increasing the rate slowly until a sudden drop in pressure is observed, indicating that hydrofracturing has initiated.
- The grout injection rate should not exceed two cubic feet per minute (cfm) and should be controlled in a manner such that rapid pressure build up can be prevented during injection, and consequently the volume of grout injected can be maximized.
- The injection rate may be adjusted to deal with different subsurface conditions encountered. Where large voids are encountered within the rock mass, it may be appropriate to temporarily increase the grout injection rate until the void is filled. However, the two-cfm limit should never be exceeded during injection in the overburden, or near the rock surface (e.g. within five feet of top of rock, etc.) where hydraulic fracturing and heave are concerns.
- During grouting, the injection rates and pressures should be monitored and recorded on the grout logs. The amount of confinement that is provided by the soil, the sensitivity of nearby structures, and pumping rate can affect the amount of grout injected.
- The injection rate may also be adjusted based on the results of any verification testing.

C.4.5 Drilling Termination Criteria

Establish appropriate drilling termination criteria based on the project objectives. For karst applications, drill at least ten feet below the apparent top of rock elevation to install the riser into bedrock. The grout holes should terminate in rock a minimum of two feet beyond any voids or soil seams. It is recommended that holes be terminated only when at least four feet of the final ten feet of the hole is drilled through rock material to ensure competent rock is encountered, and not pinnacles or floating boulders.

Subsequent series holes should be drilled to at least a depth equal to the lowest stage grout take of 25 cubic feet or more of an adjacent hole from the previous series. This requirement is especially important in karst grouting applications, where the variability and pinnacled nature of the rock may cause certain zones to be missed when only using general top of rock termination criteria. A maximum grout hole depth should be established based on the project objectives and project geology.

C.4.6 Grouting Refusal Criteria

To determine if a grout stage is complete, refusal criteria needs to be implemented. The following are general recommendations for refusal criteria and may need to be adjusted on a project-by-project basis. It should be noted that all pressure measurements must be made at the top of the riser.

- Grout flow ceases at a maximum injection pressure reading of at least 600 psi for depths greater than 30 feet below the ground surface, and 400 psi for depths shallower than 30 feet below the ground surface.
- Ground or structure movement is detected in excess of 0.01 feet, or any tilt of a surface structure is detected. Stop pressure grouting in a hole if cumulative ground heave in excess of 0.04 feet (1/2 inch), or cumulative surface structure or pavement movement of 0.02 feet (1/4 inch), is observed. The remainder of the hole should be backfilled with enough grout to fill the hole, but no additional grout should be placed under pressure.
- A volume of more than 12 cubic feet of grout is injected in each stage at a pressure of 150 psi or greater.
- A volume of more than 100 cubic feet of grout is injected in a stage. If the grout take in two consecutive stages exceeds 100 cubic feet per stage, the grouting in that hole should be suspended for a minimum of 12 hours before grouting the next stage.

Refusal pressures and volumes may be reduced in areas where sensitive soils or adjacent structures exist. Refusal pressures and volumes should also be monitored in the field and adjusted due to observations made during the grouting operations.

At each grout hole, injection of grout under pressure should be terminated at a cutoff depth for pressure injection due to lack of confining pressure. The cutoff depth for pressure injection should be set to a depth that allows pressure grouting without causing heave of the ground surface. This cutoff depth for pressure injection is typically eight to ten feet below the ground surface, but may vary based on subsurface conditions, grouting equipment, and proximity of structures, utilities, or pavement. The cutoff depth for pressure injection may be modified as grouting proceeds based on the Preconstruction Test Grouting Program and observations during production grouting. As the riser pipe is being withdrawn above the cutoff depth for pressure injection, the remainder of the hole should be backfilled with enough grout to fill the hole, but no additional grout should be placed under pressure.

C.4.7 Ground Heave and Structural Movement

The ground and nearby structures must be continuously monitored while pumping grout. The movements should be monitored near the riser location and within a minimum radius of 15 feet from the injection point, or to whatever distance may be required to monitor and prevent uncontrolled heave or damage to existing structures.

The monitoring should include a minimum of three points on the ground surface within two, eight, and fifteen feet from the injection point. Additional points may be required to prevent damage to pavements, foundations, utilities, conduits, or other structures. All points must be continuously monitored during grout injection.

Monitoring equipment such as a tripod mounted rotating laser with audible and visual capabilities, or other approved means, should be used for continuous monitoring. The equipment should be capable of accurately measuring movements of 0.01 feet or less.

C.4.8 Grouting Sequencing and Staging

In general, the sequencing should follow split-spacing between subsequently injected sets of holes. The first set of holes (primary holes) should be drilled and injected at a predetermined distance that exceeds the anticipated average reach of the injected grout. Typically, each row of primary grout holes should be sequenced so that grouting proceeds from each end of the row towards the center of the row. In addition and where practical, it is recommended that perimeter rows of a treatment area should be injected first and then grouting should be continued inwards towards the center of the treatment area isolating the area to be improved. This type of grouting sequence provides a confining perimeter zone around the treatment area and consequently helps prevent excessive grout waste and migration outside the intended treatment area. A typical split spacing hole pattern is included as Figure C.4.8-1.

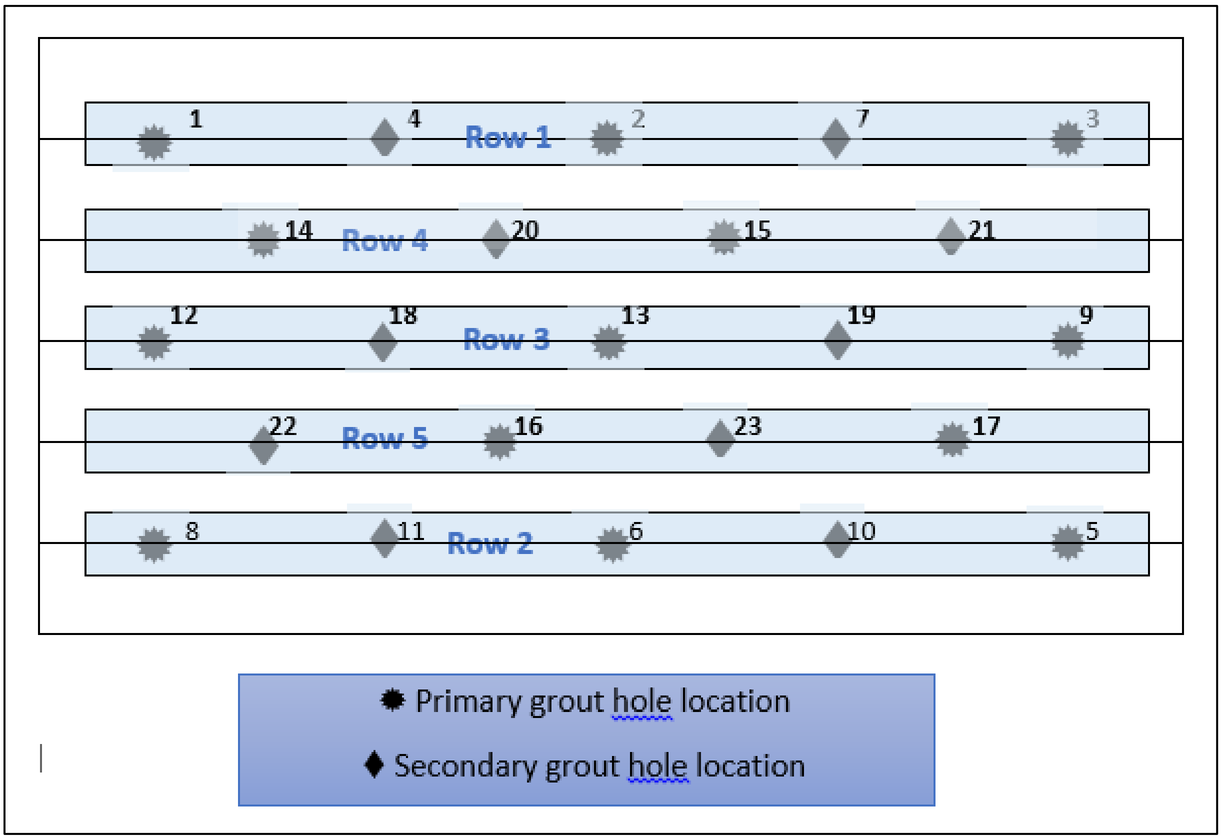


Figure C.4.8-1 – Typical Split-Spacing Hole Pattern (ASCE G-I 53-10)

The next set of holes (secondary holes) should be injected in the middle of the spacing of the primary holes. The effectiveness of LMG is typically enhanced by planning for both primary and secondary holes to be drilled and grouted at the onset of the project. The secondary hole locations have densified soil around them, caused by grouting of the primary holes, that increases the effectiveness of LMG in secondary holes. Therefore, the injected grout taken in the secondary holes is likely to decrease. It should be noted that the secondary holes should not be drilled until after adjacent primary holes have been grouted and any planned verification testing has been performed and evaluated. Secondary holes may be deleted if results from surrounding primary holes indicate low grout takes.

As grouting of the primary and secondary holes progresses, the results of the grouting with respect to grout take per hole and grout take per stage should be reviewed daily to evaluate any concerns pertaining to grout takes and procedures. Guidelines for the collection and review of drilling and grouting results is provided in Section C.6. The requirement of tertiary grout holes between the primary and secondary grout holes should be based on the developed closure criteria where adjacent primary and secondary holes have encountered takes of between 25 to 50 cubic feet or more for any stage or as outlined in Section C.4.11. Like secondary holes, the tertiary holes should not be drilled until after adjacent primary and secondary holes have been grouted and any planned verification testing has been performed and evaluated.

The length and progression of individual grout injection stages should be given careful consideration as it affects both the cost and quality of work. Typically, staging should proceed from the bottom up (upstage) with the bottom of the first stage being the bottom of the drilled grout hole. Shorter stage lengths generally provide higher quality in comparison to longer stages and should be used where rock or soil quality is poor, in particularly sensitive areas, as well as in holes where numerous voids have been identified. Typically, the grout should be injected in two-foot stages, starting from the base of the hole and proceeding to the cutoff injection depth. The grouting should advance by extracting the riser to the required stage length just before grouting each stage.

If soil densification within soil at depths of less than 15 feet is needed, top down (downstage) injection can be used. Downstage injection allows higher pressures to be used at shallow depths resulting in greater densifications as the grouted stages at the top provide additional confinement to those below. In downstage LMG for soil densification, each stage must be allowed to harden before the next is injected, resulting in a much slower and less economical process than upstage injection. A combination of upstage and downstage injection can be used where both shallow and deep densification is needed.

C.4.9 Sensitive Structures and Utilities

LMG may affect nearby structures and utilities by applying loads and inducing movements. LMG as covered by this document is intended to densify soil or fill voids without affecting nearby structures. When grouting in the vicinity of structures and utilities, the injection rate should be closely monitored and should remain low. Slower injection rates result in a greater quantity of grout being injected before movement occurs.

Movements can occur in all three dimensions so LMG can affect buried structures and utilities as well as structures above the grouted area. Movement is usually localized around the injection point. However, in karst areas, solution features containing loose soils or voids can be oriented at an angle, resulting in impacts at some distance from the injection point. Any sensitive structure or utility where movement is a concern and that are located within a reasonable distance to the injection points should be monitored for potential movement during the LMG program.

Movements should be limited per stage and on a cumulative basis. The goal is to limit movement to an elastic, "recoverable," range, and to an amount that will not cause damage. Recommended values for maximum movement for stage refusal and cumulatively are given in Section C.4.6.

Top down grout injection, as described in Section C.4.8, can be used to limit the cumulative surface effects where densification of shallow depths is needed. Top down grouting is a costlier process because of the need to allow the injected grout to obtain initial set and redrilling the grout holes after each stage.

C.4.10 Preconstruction Test Grouting Program

A preconstruction test grouting program should be used to evaluate the Contractor proposed production LMG equipment and procedures and to determine if grout hole spacing, grout pressures, and grout injection rates could be adjusted to improve the result. Before the start of any production LMG, the test grouting program should be performed using the proposed limited mobility grouting equipment and procedures. A full, independent test program may not be warranted for small projects. However, a review and evaluation of the data obtained from the first few grout holes should always be performed to evaluate and optimize procedures.

The following items constitute grout behavior that is unacceptable:

- Grout that is insufficiently mixed or grout that is not pumpable
- Grout that dry-packs in the hoses or riser pipes
- Grout that exceeds the required slump
- Grout that extrudes around the riser pipe annulus at the ground surface
- Grout that does not exhibit a stiff, mortar-like consistency
- Grout that hydrofractures the overburden soil mass, as indicated by a steady rise in grout pressure with injection followed by a sudden drop-off in injection pressure.

In addition, because the soil may not be uniform and variations in soils reaction to LMG may not be always be predictable, risk pertaining to grout finding its way to an unintended location should be taken into consideration. Careful observation and documentation of the grout rheology, pumping rate, and pressure behavior during injection should always be performed to reduce the potential for errant grout travel and to ensure proper performance.

If the results of the preconstruction test program are not satisfactory, adjustments are to be made by the Contractor (at no additional cost to the Department) to the proposed grout mix, equipment, and/or procedures to obtain the desired/specified results and acceptance criteria in the specification.

In general, the scope of an independent preconstruction grouting test program should be as follows:

- Determine line loss by measuring the pressure necessary to pump the grout at 1.5 cfm from the top to the end of riser pipe with the various anticipated lengths of grout hose and riser pipe needed for production grouting. The line loss calculation must be provided to the Department.
- Install a minimum of four limited mobility grout columns at a location near the proposed work, outside of the limits of the proposed grouting treatment area. The test column locations must be a minimum of ten feet from any production hole, and the proposed project spacing for primary grout holes must be maintained between adjacent test grout columns.
- The four columns must be installed using the design grout mix and drilling and grouting equipment to be used for the production grouting work.
- The test columns should be installed to the design depth below ground surface using specified stage heights and grout injection termination criteria.

C.4.11 Closure Criteria and Additional Holes

A LMG Program is an iterative process of injection, evaluation, and additional injection as needed. Zones that require additional treatment are identified during drilling and grouting operations. This is because the results of the drilling and grouting operations are themselves verification/evaluation holes. The larger the take per stage or series of stages, the more likely the zone requires additional treatment. The basis for determining the need for additional holes or series of holes should be defined by the Designer in the Closure Criteria section of the Special Provisions. The closure criteria are defined as the maximum allowable injection volume for each stage per hole over-which an additional adjacent LMG hole is required. Typical injection values that may require additional holes or hole series for a minimum 5-foot center-to-center LMG grid pattern range from 25 cubic feet per two-foot stage to as high as 50 cubic feet per two-foot stage, depending on the soil/rock characteristics and the program goals. The Contractor should provide recommendations for additional holes to the Department based on the defined closure criteria and on their experience and observations during the grouting operations. However, additional holes beyond those that are explicitly identified for construction on the Contract Drawings must only be allowed at the discretion and approval of the Department. Additional holes may also be added by the Designer based on a more global review of the grouting results. Areas identified as voided, soft, or high take zones during the drilling and grouting operations may have additional holes installed at the discretion of the Designer or Department to further treat and evaluate those zones.

Additional LMG holes may be required by the Designer or Department based on the results of the Verification Testing Program. However, verification testing is meant to be a check at the end of the grouting program to show adequate improvement of the site soils and/or rock and to verify program goals were achieved. Larger projects may be able to perform verification testing in completed areas while LMG operations continue in other areas. Although the results of the drilling and grouting operations usually identify the areas that require additional holes and treatment, the Verification Testing Program may identify zones not previously discovered during the LMG operations. Due to the anticipated variation in site soils both before and after grout treatment, specific verification test results are not explicitly stated in the Special Provisions or in these Guidelines. Verification holes that are advanced through LMG or soil mixed with LMG will likely exhibit favorable to exceptional results. Conversely, verification holes advanced through displaced soils or lightly treated zones between production holes may exhibit less favorable or poor results. As such, the results of the Verification Testing Program should be reviewed and evaluated accordingly. If specific zones that require additional treatment are identified during review of the Verification Testing Program, additional holes and treatment should be required at the discretion of the Designer or Department. The Contractor should also provide recommendations for additional holes to the Department based on their observations during the Verification Testing Program. Additional holes are subject to approval by the Department.

C.4.12 Verification Testing

To effectively evaluate the success of a LMG program, planning of the verification testing program is to take place during design. Before grouting the following should be well defined:

- Existing conditions
- Problem to be remediated/Goal(s) of the program
- Results that confirm grouting effectiveness

Perhaps the most important method of verifying controlled injection of low mobility grout is continuous observation of the injection rate and pressure behavior during production grouting. Pumping parameters should be immediately adjusted when loss of control of injection (hydrofracturing) is observed. Hydrofracturing is clearly indicated by a sudden drop in injection pressure following a steady rise. The pressure drop is a signal to immediately reduce the injection rate and evaluate the grout mix.

Additionally, the volume of grout injected per stage before refusal should also be observed and evaluated. If grout takes in subsequent secondary or tertiary holes are lower than grout takes in primary holes, it is an indication that the grouting program is having success. For small or reasonably straightforward projects, if the observations of injection rate and pressure behavior provide a comfort level that hydrofracturing is not occurring, and grout takes per stage are lower for secondary and tertiary holes, field observation may be enough to verify grouting effectiveness.

Verification testing defined in the LMG program for ground densification should provide a comparison of in-situ soil density before and after grouting. Typically, indirect methods such as Standard Penetration Testing (SPT) or Cone Penetrometer Testing (CPT) are used. There must be data on existing conditions before grouting to provide a basis for comparison. Reported improvements from LMG for ground densification are a three- to five-fold improvement in the SPT N-value (up to N=25 bpf) and CPT penetration resistance increases from eight to fifteen MPa. The basis for comparison of pre-grouting results versus post-grouting should depend on the relative soil density required for the intended design. For example, the improvement required for a grouted zone underlying a pavement may not be as great as the improvement required for a grouted zone underlying a load bearing structure foundation.

The advantages of SPT borings for verification testing are their common use in site explorations, hence the availability of existing data, and the ability to penetrate hard or dense material. Disadvantages include being relatively time consuming and providing data only at a small, localized area. The advantages of CPT are the relative speed and low cost of obtaining data. However, CPT also provides data only for small, localized areas and CPT cannot penetrate very hard or dense materials such as rubble, large gravel and hardened grouts.

The success of LMG for filling voids can be evaluated by methods that verify grout location, such as test excavations and SPT/core borings. Verification samples and cores can be inspected for the presence of grout in order to verify and estimate the extent of void filling. Phenolphthalein is an indicator of alkaline-base conditions and can be used to identify the presence of grout in a sample. A solution of phenolphthalein and alcohol is applied on a sample (such as a core sample). If portions of the sample turn dark pink to purple the presence of grout is confirmed.

To prepare phenolphthalein pH indicator solution, use the following:

1. Weigh out five grams of phenolphthalein.
2. Prepare a 50% ethanol (ethyl alcohol) solution consisting of 50 ml ethanol and 50 ml water.
3. Dissolve the phenolphthalein thoroughly in the 50% ethanol solution.
4. Use from a bottle fitted with an eye dropper. Store the rest in a stoppered bottle.

Geophysical methods are a possible means of providing an area-wide rather than localized evaluation of soil densification and/or void filling. Practically, they are rarely used because they require special expertise to perform and evaluate and geophysical methods need to be calibrated to other, more direct tests such as SPT or CPT. Similar to other methods of verification, a baseline geophysical survey of existing conditions before grouting should be performed to provide a basis for evaluation of improvement.

Verification testing should be integrated with the grouting program and other construction activities on the site whenever possible. Verification testing should not be performed before the completion of at least the primary grout holes. Verification testing is an integral part of the LMG program that is meant to confirm the final product meets the intent of the original program design. Verification testing results are used to evaluate the relative success of the program based on the goals of the LMG Program; therefore, verification testing must not be omitted or rendered unnecessary instead of adequate or exceptional record keeping.

C.4.13 Grouting Geotechnical Report

A Geotechnical Report should be prepared before construction, discussing project and site history, existing site conditions, recent site events, geologic conditions, test boring information, anomalous conditions, lab testing, grout program recommendations, and other relevant items. The report should also include documentation of sinkhole activity and repairs, utility repairs, pavement repairs, and any other emergency repair operations undertaken at the site. A comprehensive discussion of any geophysical investigation and results should also be included with the report. The recommendations for the grouting program should include, but is not limited to:

- discussion of the program objective
- limits of treatment
- estimated number of grout holes
- drilling and grouting termination criteria
- sequencing and staging of holes
- treatment of areas identified with large subsurface anomalies and near sensitive structures/utilities
- movement and heave monitoring
- preconstruction test grouting program requirements
- verification testing program requirements
- conceptual level cost estimate.

C.4.14 Estimating Grout Quantities

Before construction, the Engineer must provide conceptual plans containing an estimate of the number and location of holes, estimated drilling depths, estimated grout volumes for primary, secondary, contingency holes, and projected verification test quantities so that the Contractor has a basis for bidding. The quantity of grout injected is dependent not only on the subsurface conditions being treated but also on appropriate grout consistency and injection methods being specified and implemented properly in the field. Estimating the quantity of grout injected for LMG for soil densification can be roughly estimated as the volume reduction required to achieve an increase in density. The following equation can be used to estimate the percent displacement volume of grout (V_d):

$$\boldsymbol{V_d = \left(\frac{\gamma_f - \gamma_o}{\gamma_f}\right) 100\%}$$

where:

$\boldsymbol{V_d}$ = percent volume displacement required
$\boldsymbol{\gamma_f}$ = final desired total unit weight
$\boldsymbol{\gamma_o}$ = initial in-situ total unit weight

Typical values for grout volume for LMG for soil densification range from three to twelve percent of the volume of soil being treated.

C.5 CONSTRUCTION

The Contractor should submit a detailed work plan to the Department and/or Department's Representative for review and comment at least 15 days before initiating any construction activities. The work plan should include construction schedule and sequence, personnel qualifications, drilling methods and equipment, grouting procedures and equipment, grout mix design and design mix test results, measuring equipment to be used and calibration of measuring equipment, and movement/heave monitoring plan and instruments.

C.5.1 Drilling

Drilling equipment should be selected based on the intent and basis of the design, special conditions, and technical specifications. The intent and basis of the design refers to the goals of the overall project and specific grout treatment application method to be employed. "Special conditions" relates to project-specific and site-specific items including anticipated subsurface conditions, safety, and site constraints (e.g., access, terrain, height restrictions, property boundaries, utility locations, local construction ordinances, etc.). "Technical specifications" refers to items included in the project specifications and special provisions including equipment dimensions, materials, and completion criteria.

In general, rotary percussion drilling is the fastest most economical method for advancing grout holes in most types of soil and rock, including concrete and masonry. Rotary percussion drilling involves rapid, steady hammering on the drill bit as it is rotated. Rotary percussion drill rigs use a top-head hammer mounted to the top of the drill string within the drive block assembly, or a down-the-hole hammer mounted directly to the drill bit. Where greater drill depths (>100 feet) are anticipated, it is typically more beneficial to use down-the-hole hammer methods due to energy loss through the drill string at depth using top-head hammers. Most modern drill rigs can be fitted with either rotary or percussive drill heads or combination rotary-percussion drill heads.

Eccentric duplex drilling techniques are preferred in karst conditions so that a cased hole can be advanced through boulders and pinnacles without collapse. In environmentally sensitive areas or areas of limited access, it may be advantageous to advance grout holes through soil using smaller, lower energy equipment such as hand operated pneumatic drills or driven casing techniques.

C.5.2 Materials

C.5.2.1 On-Site Batching

Depending on the anticipated volume of grout to be injected on a given project, site constraints (available space, traffic, etc.), and the availability of ready-mix grout, it may be advantageous to mix grout on site using batch- or auger-type continuous mixers.

Volumetric batch plants (portable or stationary) containing bins for the various grout ingredients and a means to transport them in a controlled manner to an attached auger- or paddle-type mixer are common in LMG applications. Water is typically supplied from an on-board tank or local supply. It is imperative that a skilled grout plant operator control the proportions of the various ingredients being delivered to the mixer via adjustable gates to ensure mix consistency and desired output. Once grout is discharged into a pump hopper, holding, tank, or drum, the grout should be continuously agitated. Disproportionate mix ingredients and inconsistent mixing can contribute to clogging of the grout lines and result in false indication of pressure.

C.5.2.2 Ready Mix Grout

The use of ready-mix grout (truck delivery) from a local, qualified supplier may be desirable if site constraints such as space limitations or environmental factors are unfavorable for on-site batching.

The type and quality of the ingredients used by the supplier must conform to the materials specified in the project special provisions and grout mix design. Grout batch tickets should be collected for each load delivered to verify quantities and materials used. Grout batch tickets are typically not considered an adequate measure of material volume injected into the subsurface for payment purposes. Ready mix grout should be placed/injected within two hours from the initial batch time, or within four hours if a Department-approved set retarder is used. Any remaining grout not pumped should be properly disposed of by the Contractor.

C.5.3 Equipment

C.5.3.1 Riser Pipe/Casing

There are many different types of drill hole/grout riser pipe (casing) available ranging from ordinary steel pipe to specially fabricated or proprietary casing systems. Riser Pipe/Casing used for grouting applications are repeatedly exposed to the inherent forces associated with installation and withdraw (i.e., driving, hydraulic jacking, jaw clamps, or extreme torque). Therefore, it is imperative that any riser pipe/casing selected for LMG applications be of proper grade and quality to tolerate the rigors of the grouting process.

The specific type of riser pipe selected for a project should be chosen by the contractor and submitted to the Department/Department Representative for review and approval. The riser pipe should be sized to allow free flow of the grout with minimal head loss with no abrupt section changes. Typical riser pipe used in LMG applications consist of flush-joint steel pipes having an inside diameter between two and four inches. Larger diameter casings tend to withstand drilling pressures better than the smaller diameter casings. Larger riser pipe also reduces grout pumping losses from friction inside the pipe. Theoretically, the grout will have a greater residence time in larger riser pipe, and there may be an increased chance of having the grout lock up inside the pipe (i.e., in hot conditions, if the grout is nearing the end of its batch time, or if there has been a lot of high pressure injection, causing water to bleed out of the mix). Larger riser pipe is also heavier and more difficult to handle. Grout in pipe larger than three-inch inner diameter may free fall, which may place an undesirable static head pressure in the hole. Numerous projects have been effectively grouted using four-inch I.D. casing, but going much larger should be avoided. Riser pipe section lengths are typically five feet or less depending on injection stage intervals and equipment handling.

Riser pipe should be installed tightly to the sidewalls of the drill hole to prevent grout flow in the annulus space during injection. Due to the pressures acting on the end of the riser pipe, LMG applications typically require some means of restraining the pipe from being lifted. This can usually be achieved by using the weight of the drill rig.

C.5.3.2 Pumps

Grout pumps are required for all pressure grouting projects; however, specific pump requirements will vary for different grout materials and applications. In some instances, grouting can be effectively completed with any one of several different types of pumps, while other applications are best executed with a specific pump type. Pump performance capability must be matched to the injection parameters specified for a given application.

LMG applications typically require a piston-type positive-displacement grout pump, with a constant controllable rate of output. The pump should be capable of injecting stiff cement grout with a slump typically between one half and two inches, at controllable rates generally ranging from 0.2 to two cubic feet per minute, at continuous pressures of at least 600 psi, as measured at the top of the riser pipe. It is typically recommended that the stroke volume and volume displacement rate of the pump be calibrated/verified by pumping into a container of known volume of one cubic foot or larger. A stroke counter or other approved volume measurement device should be used to provide a record of volume injected in each stage.

C.5.3.3 Hoses

The most common grout delivery lines are high-pressure flexible hoses, but rigid delivery systems are also used in certain scenarios. Delivery lines should be sized to provide reasonable grout flow velocity and rated to resist maximum anticipated line pressure and be reasonably resistant to internal expansion. In general, for LMG applications, two-inch diameter hose is often used, and pressures of more than 600 psi are common.

A hose assembly should be routed from the pump to the hole location so that the hose is not stretched, compressed, or kinked to ensure proper flow, minimize wear and back pressure, and operate safely.

C.5.3.4 Fittings

As with all elements of the grout delivery system, couplings and connections must be able to withstand the rigors and pressures of grouting. Grout mixes used in LMG applications are designed to flow through the delivery lines as a stiff extrusion. A common problem associated with LMG applications is internal expansion or swelling of the flexible hose lines. An expanded grout mass or extrusion encountering the more ridged couplings and connections can result in pressure loss at the injection point, excessive back pressures, or plugging of the line.

All couplings and fittings in the delivery line should be watertight under pressure and have a constant inside diameter that does not protrude into the grout flow. Leaks in the delivery lines allow water to be squeezed out of the mix during flow, which can result in packing and plugging of the line. Clamp-type couplings with replaceable rubber gaskets are most often used in LMG applications.

C.5.3.5 Pressure Gauges

Pressure gauges are among the most important tools on a LMG project, as it is through the observation of these gauges that all other equipment is controlled. Gauges come in many different sizes, pressure ranges, degrees of accuracy, and quality. Gauge accuracy is defined as a percentage of the full-scale range of a specific gauge. In general, gauges should be selected such that operating pressures fall within the middle 50 percent range of a particular gauge for optimum accuracy.

Grout pressures are indirectly controlled by the operator through adjustments to the grout injection rate and position in the subsurface. The ability for the operator and inspector to read the pressure gauge is paramount. The pressure level and behavior (i.e., changes that occur during injection) should be closely monitored and recorded to facilitate effective adjustments during grouting.

For LMG applications, the contractor should provide new pressure gauges with capacities of zero to at least 600 psi, but no greater than 1,000 psi, and a minimum face diameter of three inches for readability. Pressure gauges should be marked in 10 psi divisions for the full scale, with a minimum accuracy of one-half division. Certificates for calibrations for each gauge, performed within six months before construction takes place, should be submitted before using the gauge. Any gauge of questionable accuracy must be promptly replaced. Enough gauges to cover replacement and recalibration without any delay in work should be provided. At a minimum, gauges should be placed at the top of the injection riser and grout pump. To prevent damage, the gauging elements should always be kept clean and in mechanically-sound condition.

C.5.4 Grout Measurement and Calibration

The quantity of grout injected is typically determined by the grout pump stroke count. The stroke count refers to the physical number of times a grout pump cylinder cycles from fully open to fully closed over one grout stage (ramming out one stroke of grout). Due to the variable complexities of the grouting process, grout cylinders commonly do not fill completely resulting in discrepancies between the actual grout volume extruded per cylinder stroke and the theoretical volume of the cylinder. To better define the actual volume of grout extruded per cylinder stroke, it is practical to calibrate the pump output by filling a box of known volume (one cubic foot) before daily grouting operations or any time a change in output may have occurred. The approximate quantity of grout pumped over a given period of time (i.e., per hour, per day, or per hole) based on stroke count should be checked against the quantity of materials used (i.e., batches, trucks, or loads), and appropriate adjustments made to best represent actual quantity injected. It should be noted that any grout waste such as grout remaining in the delivery lines and the hopper or grout flushed out of extracted riser pipe be subtracted from the total quantity of material used. It is imperative that grout quantity measurements be as accurate as possible where injection quantity is the basis for payment.

C.5.5 Movement Monitoring

Monitoring of the ground surface and any structures in the vicinity of grouting operations for movement is a crucial part of any grouting program. In addition, it is important to consider the three-dimensional nature of movements in the subsurface as they can affect underground utilities, retaining walls, footings, and other buried structures. Monitoring of surface movement should include vigilant visual and instrument observations. It is essential to maintain direct lines of communication between the monitoring points and the pump operator to reduce the likelihood of undesirable movements that may result in the decreased efficacy of the grouting program.

Contractors should provide the equipment necessary to detect movements of the ground surface or structures including buried utilities using tell-tale indicators within or near the work zone. Typical monitoring equipment consist of rotating laser levels capable of accurately measuring movements of 0.01 feet or less and equipped with audible and visual alarm indicators. Monitoring for heave at the ground surface and nearby structures should be performed at a minimum of three points for each injection location.

C.5.6 Pressure Losses

Pressure losses occur as the grout is pumped through the grout hoses and fittings and the grout riser pipe. Measuring pressure at the top of the riser pipe will eliminate the pressure losses through the grout hoses and fittings from consideration, but there will still be pressure losses between what can be practically measured at the top of the riser pipe and the point of injection at depth. Before grouting, pressure losses can be measured by laying a length of riser pipe on the ground surface and pumping grout through it at a rate comparable to anticipated injection rates. This measurement gives an offset pressure that can be used to relate the grout bulb pressure to the pressure measured at the header. A calculated pressure head from the weight of the grout to the injection stage should also be incorporated. With deep holes, there can be some errors with this approach because the pressure in the grout near the surface differs from that at depth which results in a different saturation and void ratio. However, because pressure losses are small compared to injection pressure, the approach of using a pressure offset between the header and the injection point is reasonable.

C.5.7 Quality Control

Quality control for all operations must be established and maintained by the Contractor to ensure compliance with specifications, and records of quality control must be maintained for all operations. As part of the quality control, the contractor is to ensure that satisfactory drilling and grouting equipment is provided and kept in good mechanical condition, that the work complies with all requirements of the specifications, and that work areas are protected and properly cleaned up. Contractor and Department field personnel for a grouting operation should be aware of the purpose and mechanism of the proposed drilling and grouting and methods for measuring compliance so that they can identify poor conditions and propose adjustments to methods and materials when they are encountered. A standard special provision, providing additional detail regarding Contractor QC requirements is included in the Department Engineering and Construction Management System (ECMS).

The contractor is responsible to monitor the following and provide well-ordered and intelligible records to the Department in a timely manner for review:

1. Drilling, including:

    a. Hole location and ground surface elevation
    b. Total depth drilled/depth riser pipe installed
    c. Depth of soil overburden
    d. Depth to top of rock
    e. Penetration rate and/or drilling time
    f. Possible voids, fractured zones, and other conditions noted during drilling
    g. Drilling fluid communication between the drill hole and any adjacent holes that have not yet been grouted
    h. Zones of possible voids or soft soils
    i. Fractured rock and soil zones below top of rock

2. Grouting, including (for each stage/total, as appropriate):

    a. Stage depths
    b. Pumping rate
    c. Stroke count
    d. Volume of grout placed
    e. Pressure
    f. Refusal criteria
    g. Heave of ground surface
    h. Tilt/Heave of structure
    i. Sudden drop in pressure
    j. Any changes to grout mix

3. Verification Testing (as required for the type of testing specified):

    a. SPT driller's logs
    b. CPT driller's logs
    c. Report documenting geophysical testing

C.5.7.1 Grout Testing

Before construction, a grout mix design to achieve the specified strength, pumpability, consistency, and slump must be submitted by the Contractor and approved by the Department. Take cylinder samples for testing daily, and when the grout mix changes. If ready-mix grout is being used, truck tickets must be provided and cylinder samples must be taken for each truck. Perform slump testing for each batch of grout, at the pump hopper, with a minimum of one test per ten cubic yards of batched material.

C.5.7.2 Inspection

Active construction monitoring by the Department Representative serves not only as Quality Assurance, but also is a key component in modifying the proposed LMG program to maximize its effectiveness through observational methods as a basis for field decisions. The Department Representative should be present for all drilling and grouting. Monitoring and documentation by the Department Representative should include the items listed in Section C.5.7. An example Drilling and Limited Mobility Grouting Field Log is included as Figure C.5.7.2-1. The Department Representative should be responsible for confirming the depth at which the grout holes are terminated according to the criteria presented in Section C.4.5 and the occurrence and type of grout refusal per stage, as determined according to the criteria presented in Section C.4.6.

DRILLING AND LIMITED MOBILITY GROUTING FIELD LOG

REV. 3/17

pennsylvania DEPARTMENT OF TRANSPORTATION

S.R.: ____________ **Section:** ____________ **County:** ____________ **Hole No.:** ____________

S-Number: ____________ **Substr. Unit:** ____________ **ECMS:** ____________ **Page:** ______ of ______

Baseline: ____________ **Station:** ____________ **Offset:** ____________ **Ground Elev:** ____________

Operator: ____________ **Date/Time**

Company: ____________ **Start Drilling:** ____________ **Drilled Depth:** ____________

Inspector: ____________ **End Drilling** ____________ **Casing Depth:** ____________

Drilling Method: ____________ **Start Grouting:** ____________ **Casing I.D.:** ____________

End Grouting: ____________ **Hole Dia:** ____________

Grout Mix: ____________ **Slump:** ____________ **Daily Pump Calibration Factor:** ____________

Depth (ft)	Soil/Rock Material Description	Hammer Rate	Time of Day (Grouting)	Injection Rate (cfm)	Injection Pressure Range (psi)	No. Pump Strokes	Grout Placed (cf)	Grout Stage Refusal Criteria	Drilling and/or Grouting Comments

Hammer Rate	GROUT TAKE TOTALS (Strokes & Volume)				Grout Stage Refusal Criteria
I = Intermittent S = Steady N = No Hammer		THIS PAGE ONLY			PS = pressure surge VC = volume cutoff PV = pressure & volume
		TOTAL FOR HOLE			HS = heave at g.s. GS = grout at g.s.

Figure C.5.7.2-1 –Drilling and Limited Mobility Grouting Field Log

C.6 DATA MANAGEMENT

Records that must be kept and maintained during the grouting program include grout hole location plan, drilling and grouting logs (per hole), groundwater level measurements (if appropriate), grouting pressures, movement monitoring and instrumentation reports (if separate), observations during grouting, grout mix or batch logs, and QA/QC testing results. Records must be saved electronically in a format that is readable by standard Department software and ordered for easy access and reference. PDF copies of handwritten logs are acceptable provided the information presented is neat and legible. Records must also reflect the acquired understanding of the site conditions by both the Department and the Contractor. It is the responsibility of the Contractor to keep and maintain all pertinent drilling and grouting records and submit them to the Department in a timely manner.

Contractor records should be reviewed by qualified Department personnel and/or Representative that are experienced in LMG grouting. Before the beginning of LMG work, the Department should designate the personnel that will review Contractor records and evaluate the grouting results. The designated Department Representative will also determine the need for additional LMG holes and will make those recommendations to the Department. The Department may choose to designated personnel internally or may choose to supplement oversight and review to a third party (i.e., Engineer/Designer or qualified Specialist). The Department Representative should be qualified in LMG grouting design and construction.

The grouting records should be reviewed by both the Contractor and the Department Representative to verify quality and consistency of all grouting operations as stated in the Special Provisions, including grout mix QC test results. Verification testing and results submitted by the Contractor should be reviewed by the Department Representative to confirm that program design goals have been achieved. The Contractor should submit their recommendations for additional holes and verification testing to the Department. The Department will perform an independent review and determine if the program design goals have been achieved or if additional grouting operations are required. Frequent communication and consistent feedback between the Department and their on-site Representative is essential for Quality Assurance and project verification.

All records should be reviewed for completeness and accuracy by the Department Representative daily. Any deficiencies should be alerted to the Contractor immediately. Daily reviews should be performed by the Department Representative at the beginning of the project in order to confirm the records are being generated and collected in an appropriate, consistent, and timely manner according to the Special Provisions, and to confirm there are no obvious deficiencies in how the grouting program is being progressed. Any apparent deficiencies in the records should be alerted to the Contractor immediately for resolution. Review frequency may be reduced by the Department Representative depending on the size and pace of the work only after the Department is comfortable with Contractor means and methods. The level and frequency of review by the Department Representative should be agreed upon by both the Department and their Representative.

As grouting progresses, the grout takes per hole and grout takes per stage should be reviewed by the Department Representative daily to evaluate any concerns pertaining to grout takes and procedures, or to identify the need for additional holes. The Contractor should also review the results and submit their recommendations for additional holes. However, the Department will make the final decision if additional holes are required. The amount of data that is generated by grouting operations will quickly become overwhelming and decision making will be difficult without good data management. Data management should consist of both a summary spreadsheet and a working grout plan. The spreadsheet and plan can vary in detail depending on the size and complexity of the project but should be frequently updated with field data. At a minimum, the data should include the following:

- Grout takes per stage (by depth) and total grout takes
- Grout hole location and depth
- Depth to top of rock
- Depth and limits of voids or critical zones identified for treatment
- Location of exploratory borings
- Location of verification borings

Color coding of grout takes by ranges of magnitude is a straightforward way to visually illustrate areas, depths, and concentrations of high or low volumes of grout take for the purpose of determining where tertiary holes are necessary, or where secondary holes are not needed. The use of secondary and tertiary grout holes is discussed in Section C.4.8.

RESERVED FOR
FUTURE DEVELOPMENT OF:

PUBLICATION 293
GEOTECHNICAL ENGINEERING MANUAL

APPENDIX D – ROCK MASS RATING (RMR)

RESERVED FOR
FUTURE DEVELOPMENT OF:

PUBLICATION 293
GEOTECHNICAL ENGINEERING MANUAL

APPENDIX E – DESIGN/CONSTRUCTION IN KARST

RESERVED FOR
FUTURE DEVELOPMENT OF:

PUBLICATION 293
GEOTECHNICAL ENGINEERING MANUAL

APPENDIX F – GROUND MODIFICATION

RESERVED FOR
FUTURE DEVELOPMENT OF:

PUBLICATION 293
GEOTECHNICAL ENGINEERING MANUAL

APPENDIX G – SUBSIDENCE