Geotechnical Design Manual

2008

Version 1.0

Issued in August 2008

PREFACE

The *South Carolina Department of Transportation* (SCDOT) *Geotechnical Design Manual* ("Manual") has been developed to provide uniform design practices for SCDOT and consultant personnel preparing geotechnical reports and contract plans for SCDOT projects. The purpose of this manual is to complement the Mission of SCDOT by providing for safe, economical, effective and efficient geotechnical designs. [1]

The designer should attempt to meet all criteria and practices presented in the Manual, while fulfilling SCDOT's operational and safety requirements. SCDOT Pre-Construction Support - Geotechnical Design Section (PCS/GDS) should be consulted when deviations from the guidelines presented in this Manual are needed. The Manual supersedes all previous editions or publications relating to the geotechnical aspect of transportation projects.

The Manual presents most of the information normally required in the geotechnical design of transportation projects; however, it is impossible to address every situation that the designer will encounter. Therefore, designers must exercise good judgment on individual projects and, frequently, they must be innovative in their approach to geotechnical design. This may require, for example, additional research into geotechnical literature.

The July 2008, Version 1.0, of the Manual consists of Chapters 1 through 12 and is being issued prior to the completion of the entire Manual. These Chapters reference Chapters that have not been written. In these cases, the geotechnical engineer, either in-house or consultant, shall use the current state-of-practice for those Chapters referenced. Any questions concerning the applicability of procedure, analysis, or method should be directed to the PCS/GDS for review and comment. As additional Chapters are completed, these Chapters will be added to the Manual.

[1] SCDOT's Mission is as follows: *"The department shall have as its functions and purposes the systematic planning, construction, maintenance, and operation of the state highway system and the development of a statewide mass transit system that is consistent with the needs and desires of South Carolina citizens. The goal of the department is to provide adequate, safe, and efficient transportation services for the movement of people and goods."*

Table of Contents

Chapter 1

INTRODUCTION

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Table of Contents

List of Figures

CHAPTER 1

INTRODUCTION

1.1 INTRODUCTION

This Chapter presents the responsibilities of the Geotechnical Design Squads (GDSs) within the *South Carolina Department of Transportation* (SCDOT). The GDSs are responsible for providing geotechnical engineering expertise in the areas of planning, design, construction, and maintenance for South Carolina's bridges, roadways, and other transportation related structures and facilities. Geotechnical engineering is defined as the investigation and engineering evaluation of earth materials including soil, rock, groundwater, and man-made materials and their interaction with structural foundations, earth retaining structures, and other civil engineering works. General guidance is provided in this Chapter with reference to the geotechnical engineering services that the GDSs provide the SCDOT. Chapter 2 describes the geotechnical project coordination process within the Preconstruction phase of project development. Together, Chapters 1 and 2 provide the reader with an understanding of the necessary interaction among the various Units in coordinating the geotechnical involvement in typical road and bridge projects.

The GDSs perform design related services including development of field explorations and construction support. For design, the GDSs coordinate with the Office of Materials and Research in obtaining field and laboratory tests. In addition, the GDSs prepare bridge and roadway geotechnical reports for use by the Structural and Road Design Groups. Further, the GDSs review reports prepared by Consultants for technical content, and compliance with this Manual. The GDSs also review plans prepared by both the Structural and Road Design Groups as well as plans prepared by Consultants to assure that the geotechnical information provided has been properly interpreted. The GDSs also provide support to the Construction Office in review and acceptance of Contractor geotechnical submittals.

The following sections describe the geotechnical engineering services that the GDSs provide to the Preconstruction Division, SCDOT Units external to the Preconstruction Division, and to agencies outside of SCDOT.

1.2 PRECONSTRUCTION DIVISION

The Preconstruction Division is divided into 7 subdivisions. Four Regional Production Groups, a Preconstruction Support Group, the Right-of-Way Office, and the Surveys Office. The GDSs are part of the Structural Design Groups within the Regional Production Groups (RPGs). In addition, a GDS is also located within the Preconstruction Support Group (PCS/GDS). During the development of road and bridge design projects, the GDSs will coordinate the geotechnical subsurface investigation and then issue geotechnical reports with design recommendations to the Structural and Road Design Groups within each RPG.

1.2.1 Regional Production Groups

The RPGs provide engineering and project management for projects located within specific geographic areas of South Carolina. Figure 1-1 provides the geographic boundary of each RPG.

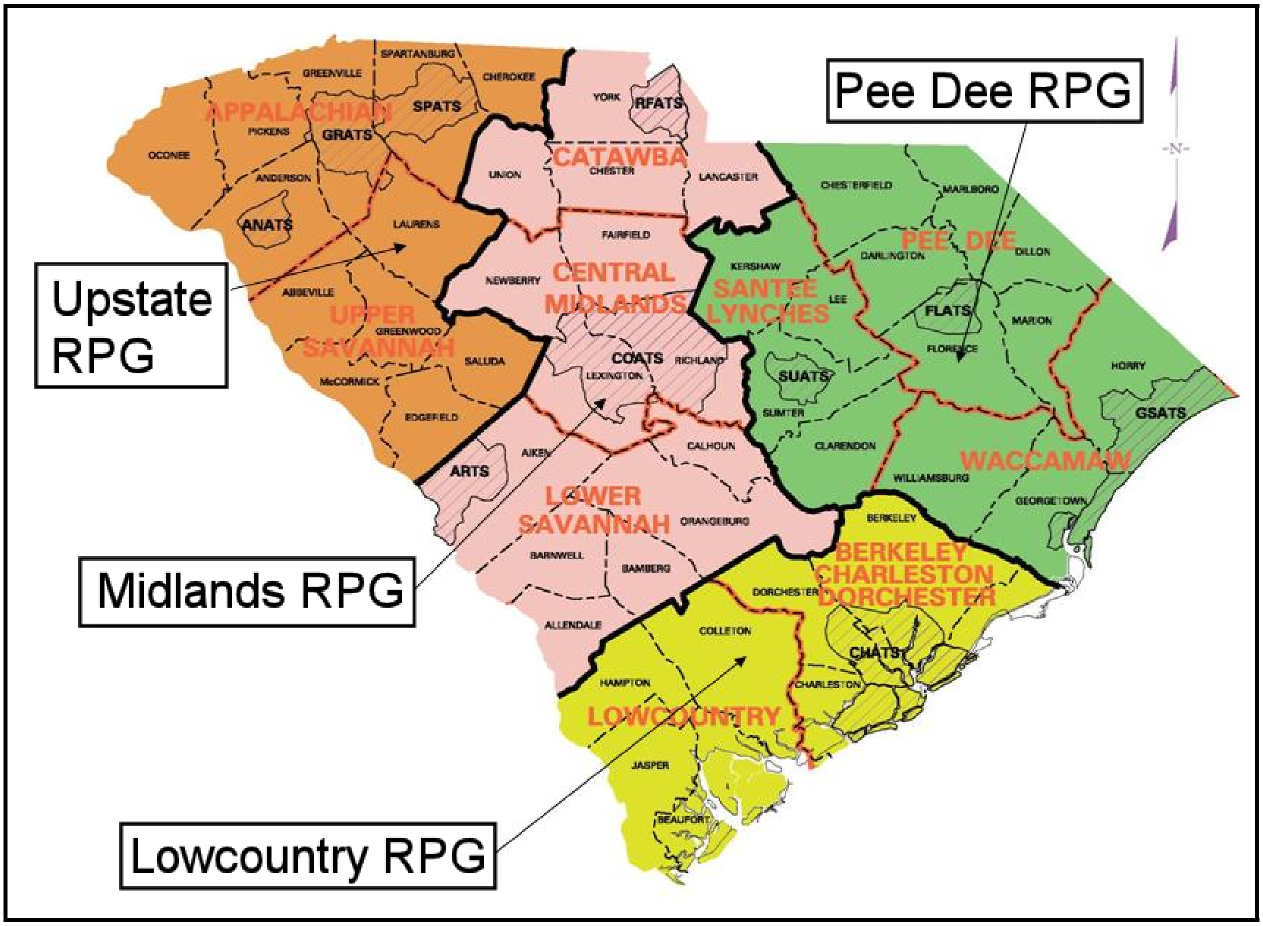


Figure 1-1, Regional Production Groups

Each RPG consists of Program Development, Road, Structural, Hydraulic and Utilities Engineering Groups. The Road, Structural, Hydraulic and Utilities Engineering Groups report to the Design Manager, the Design Manager has overall responsibility for coordinating project designs. The GDS is part of the Structural Design Group, which is also comprised of Bridge and Roadway Structures Design personnel. The geotechnical services that the GDS provides the other squads within each RPG are described below.

1.2.1.1 Program Development

The GDS will work closely with Program Development and C Projects (non Federal-Aid) by being included in the Project Development Team on all projects that may require geotechnical design. The GDSs primary responsibility as part of the Project Development Team is to provide geotechnical expertise in all phases of the project development process. As part of the Project Development Team, the GDS coordinates the geotechnical subsurface investigation and provides geotechnical guidance and geotechnical designs with respect to:

- Roadway Alignment
- Roadway Structure Foundations
- Earth Retaining Structures
- Roadway Embankment Design
- Bridge Foundation Design
- Project Staging
- Geotechnical Consultant Review

The GDS provides its input by attending pre-design meetings, Project Development Team meetings, and participating in Design Field Reviews (DFR). In addition to these meetings, the GDS prepares two Preliminary Geotechnical Engineering Reports (PGER), one for the road and one for the bridge, Roadway Geotechnical Engineering Reports (RGER), and Bridge Geotechnical Engineering Reports (BGER), and geotechnical memoranda as needed.

1.2.1.2 Road Design

The GDS is responsible for developing a soil exploration program and preparing a PGER and RGER. The PGER provides general geotechnical recommendations based on limited soils information obtained from existing soil information and the preliminary subsurface investigation. The general geotechnical recommendations, from the PGER, are used to evaluate the DFR plans. After the DFR has been conducted, a detailed subsurface soil exploration program is conducted based on the required structures defined during the DFR. The RGER provides design recommendations for roadway earthwork and roadway structures. Roadway earthworks such as cut excavations and fill embankments are evaluated for stability and performance. Earthworks are designed under static and seismic loading conditions to meet the geotechnical design criteria presented in this Manual. The RGER is provided to the Road Design Group for inclusion of the GDS recommendations in the plans and specifications. The GDS provides stability (global, bearing capacity, sliding, etc.) and settlement analysis for fill embankments and cut sections. A detailed discussion of what should be included in a PGER and RGER is provided in Chapter 21. In addition to these geotechnical reports, the GDS may develop or assist in development of specifications and special provisions (Chapter 23) pertaining to soils, rock, ground improvement methods, earth retaining structures, and foundation systems.

The GDS also reviews geotechnical engineering calculations and plans prepared by Contractors, Consultants, or Suppliers to ensure conformance with SCDOT design standards and policies.

1.2.1.3 Structural Design

The GDS is responsible for developing a soil exploration program and preparing a PGER and BGER. The PGER provides general geotechnical recommendations based on limited soils information obtained from existing soil information and the preliminary subsurface investigation. The general geotechnical recommendations may be used to recommend foundation types, perform seismic evaluations, and assist in the establishment of tentative bridge lengths. After the DFR has been conducted, a more detailed subsurface soil exploration is conducted based on the bridge spans and anticipated foundation type. The BGER is used to design foundations for bridges and bridge related structures. Bridge foundations are designed for static and seismic loadings. Bridge foundation recommendations include foundation type and size, structural design information, and plan notes for construction drawings. Bridge related structures such as wing-walls, abutment walls, MSE walls, etc. are evaluated for stability, performance, and structural design. If stability or performance of these structures does not meet the geotechnical design requirements as presented in this Manual, geotechnical design recommendations are provided to the project manager/bridge designer. Foundation recommendations include foundation type (spread footing or deep foundation), stability (global, bearing capacity, sliding, etc.), and structure performance (settlements, displacements, etc.). Foundation recommendations for roadway structures such as retaining walls (fill walls and cut walls) and culverts (box and 3-sided) are provided by the GDS. Foundation recommendations would include foundation type (spread footing or deep foundation), stability (global, bearing capacity, sliding, etc.), and structure performance (settlements, lateral displacements, etc.). The BGER is provided to the Bridge Design Squad for inclusion of the GDS recommendations in the plans and specifications. The recommendations for roadway structures will be provided in either the BGER or the RGER depending on which set of plans will contain the structure (i.e. is the structure in the Bridge or Road Plans). A detailed discussion of what should be included in a PGER, BGER and/or RGER is provided in Chapter 21. In addition to these geotechnical reports, the GDS may develop or assist in development of specifications and special provisions pertaining to soils, rock, ground improvement methods, earth retaining structures, and foundation systems.

The GDS reviews geotechnical engineering drawings, geotechnical engineering calculations, specifications, and geotechnical engineering reports prepared by Contractors, Consultants, or Suppliers to ensure conformance with SCDOT design standards and policies. When the Contractor is responsible for designing a roadway structure (i.e. MSE wall, soil nailing, etc.) during construction, the Contractor is required to provide a geotechnical report prepared in accordance with the Manual. The report will be reviewed by the GDS for technical content and compliance with this Manual.

1.2.1.4 Hydraulic Engineering

The GDS is responsible for obtaining soil samples within potential scour zones and assigning laboratory testing for use by the Hydraulic Engineering Squad in evaluating the potential and magnitude of scour at bridge and hydraulic structures. In addition, the GDSs:

- Coordinate with Hydraulics and Structural Design Groups for bridge and culvert designs; and,
- Provide input, analysis, design recommendations, and/or review for slope protections in cases of moderate to severe erosion or erodability potential.

The Hydraulic Engineering Group is responsible for performing and/or reviewing hydrologic and hydraulic analyses on all projects for both roadway drainage appurtenances and bridge waterway openings. The responsibilities of the various engineering groups of the RPGs are as follows:

1. Survey Request. The Road Design Group is responsible for forwarding the Survey Request to the Hydraulic Engineering Group for its review and approval. This activity

generally occurs after the Program Action Request (PAR) has been prepared and routed to the appropriate personnel by the Design Manager.

2. Hydraulic/Scour Report. Any structures over a waterway require a Hydraulic/Scour Study. Once the general bridge location is known, the Structural Design Group will prepare a hydraulic request to the Hydraulics Engineering Group to conduct the necessary studies and prepare the applicable reports. Based on the hydrologic data collected and the preliminary plan and profile, the Hydraulic Engineering Group will perform the detailed hydraulic analysis for a bridge. The Report will provide the following information to the Structural Design Group:

   - The necessary bridge waterway channel bottom width, side slopes, skew angle, and channel centerline station;
   - National Pollutant Discharge Elimination System (NPDES) boundary information; and,
   - The results of the hydraulic scour analysis.

1.2.1.5 Utilities Engineering

The Utilities Engineering Group is responsible for coordinating with utility companies impacted by highway improvement projects. The Utilities Engineering Group will coordinate between the GDS and local utility companies to resolve conflicts between borings and utility locations. In addition, GDS can provide the following services:

- Trench, temporary shoring, braced excavation design, review;
- Special provisions or Supplemental Specifications; and,
- Design, review, and/or guidance on backfill for pipes, sewers, storm sewers, lift stations, etc.

1.2.2 Preconstruction Support Group

A Geotechnical Design Squad is also located within the Structures Engineering Group of the Preconstruction Support Group (PCS/GDS). The PCS/GDS will be responsible for providing Quality Assurance services for geotechnical engineering products (i.e. reports and letters) that will be used to support engineering and construction projects. In addition, PCS/GDS will also be responsible for preparing and updating this Manual and other documents that will affect geotechnical engineering design procedures. Further, the PCS/GDS will lead training efforts within the various production oriented GDSs. The PCS/GDS will develop, recommend and oversee implementation of geotechnical engineering policies and procedures. The PCS/GDS will further provide technical support to the other GDSs.

1.2.3 Right-of-Way Office

The GDS is responsible for coordinating with the Right-of-Way Office to obtain Access Permission that allows the Department to conduct a geotechnical soil exploration on properties that are currently being acquired by the State. This typically occurs when a highway project is on a new alignment or where widening of a current alignment requires the acquisition of adjacent properties. In addition, the Right-of-Way Office will provide coordination with railroad companies impacted by highway improvement projects. Railroad coordination must occur as early as practical in the project development process. The Right-of-Way Office will assist in the coordination with railroads to provide access for drilling equipment where the transportation structure crosses or is in conflict with the railroad.

1.2.4 Surveys Office

The Surveys Office is responsible for conducting aerial and field surveys for all Department projects. The Surveys Office will assist in locating all soil test-boring locations in the field both prior to and after completion of field services, if boring locations have been moved with the approval of the GDS. The Surveys Office shall obtain the approximate elevation and coordinates (latitude and longitude) of all testing locations. The Surveys Office shall provide this information to the Materials Geotechnical Engineer in the Office of Materials and Research.

1.3 SCDOT UNITS EXTERNAL TO PRECONSTRUCTION DIVISION

The GDSs also work and coordinate with other divisions of SCDOT. Listed below are the divisions that the GDSs work with:

- Planning
- Environmental Management
- Traffic Engineering Division
- Construction Division
- Maintenance Division
- District Offices

A brief description of the type of geotechnical engineering services that the GDSs provide these Divisions is provided below.

1.3.1 Planning

The Planning Office assesses the scope and cost of project alternatives for the Project Study Report. The Planning Office also works closely with Metropolitan Planning Organizations (MPOs) and Council of Governments (COGs) to develop long-range transportation plans for local areas. In addition, this office also focuses on the wider range of transportation projects, including not only highways, but also ports, railroads, and mass transit efforts. The GDSs interface with this office by providing literature searches of available geotechnical information, field reconnaissance, geologic mapping, and subsurface explorations. In addition, the GDSs may be requested to prepare geologic hazard commentary and data for use in project documents and, on request, address geologic hazard issues at public hearings.

1.3.2 Environmental Management

The Environmental Management Office is within the Planning Division and is responsible for a variety of activities related to environmental impacts and procedures. This includes air, noise, and water quality analyses; biological, archeological, and historical impacts; preparation of environmental documents for SCDOT projects; evaluation and mitigation of hazardous waste sites; and public involvement. In particular, the Environmental Management Office coordinates with the applicable Federal and/or State agencies for processing the permit information and obtaining the agency approvals. The GDSs and the Environmental Management Office will coordinate to ascertain potential environmental impacts of drilling operations. The impacts include wetland impacts of drill rig access and potentials for soil and groundwater contamination that could be a health or environmental hazard.

1.3.3 <u>Traffic Engineering Division</u>

The Traffic Engineering Division provides a variety of traffic engineering services to other Departmental Units (e.g., traffic control devices, highway capacity analyses, traffic engineering studies). Where a bridge project involves the removal of an existing structure in a specific sequence during construction, the Structural Design Group will assist the Traffic Engineering Division in the development of the proposed Work Zone Traffic Control Plans; otherwise, the Traffic Engineering Division provides the Road Design Group with the required information. The Road Design Group then provides this information to the Structural Design Group when it becomes available. The GDSs will provide geotechnical services related to traffic engineering for Headquarter and District offices by:

- Providing foundation design and/or review for signs, traffic lights, and other structures; and,
- Coordinating traffic control with temporary shoring when necessary.

1.3.4 <u>Construction Division</u>

The Construction Division, in coordination with the District Offices, is responsible for all construction activities on all State maintained roads. This includes the development of specification, inspections and staffing, and approval of construction change orders.

The GDSs provide support to the Construction Division through the Resident Construction Engineer (RCE) during construction of the geotechnical portion of projects and assists in resolving situations resulting from soils and foundation problems. The GDS will also review significant features exposed during construction to compare actual conditions to those anticipated during design, and to make corrective recommendations as necessary. If Foundation Testing is required, coordinate the testing with the RCE. The following summarizes the coordination between the GDSs and the Construction Division:

1. <u>Shop Plans</u>. Contractors are responsible for submitting the required Shop Plans (e.g., structural steel, prestressed concrete piles, MSE wall, etc.) to the RCE who then forwards the Shop Plans to the Pre-Construction Support Engineer for distribution, review and approval. See Section 725 of the *SCDOT Construction Manual* and Chapter 24 – Construction QA/QC for more details on Shop Plans.

2. <u>Installation Plans</u>. Contractors are required to submit installation plans for certain types of construction (e.g. piles, drilled shafts, etc.). The installation plans are submitted to the RCE. The RCE forwards the plans to headquarters. The responsible GDS will review the plan for compliance with the appropriate specification on special provision (see Chapter 24 for more information).

3. Temporary Structures. If temporary structures are required on a project, the contractor shall submit design drawings for the temporary structure to the RCE. The RCE will forward the designs to headquarters for review and approval. All temporary designs that involve geomaterials will be reviewed by the responsible GDS for compliance to the specification on special provision (see Chapter 24 for more information).

4. Value Engineering Proposals. The Department encourages contractors to submit Value Engineering Proposals. Upon receipt, the RCE will contact the appropriate SCDOT offices to discuss the original design intent and the potential merits and cost savings of accepting the proposal. If approved by the Department, the Value Engineering Proposal will require the creation and proper execution of a Change Order.

5. Constructability Reviews. Selected projects may undergo a constructability review to ensure that a project is buildable, cost effective, biddable, and maintainable. A representative from the Central Construction Office is the Team Leader during all constructability reviews; however, the Structural Design Group is responsible for the organization of the review.

1.3.4.1 Bridge Construction

The GDSs review all in-house and consultant pile driving and drilled shaft installation plans. Provide assistance with constructability issues relating to bridge foundations, approaches, embankments, and approach slabs.

After the awarding of construction projects, the GDSs also works closely with the Construction Division to provide geotechnical construction support services.

1.3.4.2 Road Construction

The GDSs provide assistance with constructability issues relating to subgrade preparation beneath embankments, embankments, retaining walls, culverts, temporary retaining structures, and approach slabs.

After the awarding of construction projects, the GDSs also works closely with the Construction Division to provide geotechnical construction support services.

1.3.4.3 Materials and Research

The GDSs maintain an open line of communication with Materials Geotechnical Engineer. When necessary, any subsurface field investigations, requested by the GDSs will be forwarded to the Office of Materials and Research (OMR). The GDSs will:

- Coordinate with OMR to obtain subsurface investigations; and,
- Develop or assist in developing specifications and supplemental specifications pertaining to soils, rock, and/or foundation systems

1.3.5 Maintenance Division

The GDSs evaluate chronic, urgent and emergency situations resulting from geotechnical problems, such as landslide repairs, assist in the development of plans, specifications and estimates for projects to correct such conditions. Further the GDSs set the scope of geotechnical studies for the roadway portions of projects done by consultants, work with the consultant in selecting appropriate analyses and design options, provide ongoing geotechnical review during the consultant's work, and provide general technical oversight. In addition, the GDSs:

- Provide remedial design in cases of slope or embankment failure (including settlement analysis) and/or landslide;
- Provide input, analysis, design recommendations, and/or review for slope protections in cases of moderate to severe erosion or erodability potential;
- Provide input for subsurface investigation and laboratory soil analysis for maintenance bridge replacement;
- Provide foundation design for maintenance bridges, as required;
- Assist with analysis, design, and emergency action plan input in cases of bridge failure; and,
- Provide assistance with regard to constructability issues associated with bridge foundations, approaches, embankments, and approach slabs.

1.3.6 District Offices

The SCDOT is organized into a Headquarters and 7 Districts. In each District there is a District Engineering Administrator (DEA) that oversees the operations of the District Construction, Maintenance, and Traffic Engineering personnel. The GDSs provide geotechnical engineering support to the District Construction, Maintenance, and Traffic Engineers. The GDSs typically provide geotechnical engineering through the Headquarters coordinator for Construction (Bridge or Road), Maintenance, or Traffic Engineering.

1.4 FEDERAL HIGHWAY ADMINISTRATION

The Federal Highway Administration (FHWA) administers the Federal-aid program, which funds eligible highway improvements nationwide. Its basic responsibility is to ensure that the State DOTs comply with all applicable Federal laws in their expenditure of Federal funds and to ensure that the State DOTs meet the applicable engineering requirements for their proposed highway projects. FHWA maintains a Division Office within each State, and this Office is the primary point of contact for a State DOT.

The GDSs routinely confer with the following FHWA office regarding the following:

- SC Division Office: Complex geotechnical designs, geotechnical policies, specifications, supplemental specifications;
- Office of Bridge Technology – Geotechnical Engineering: Review of new procedures and completed designs; and
- Resource Center – Geotechnical and Hydraulic: Obtain new technologies that could impact projects. The impacts include reducing construction times and saving money.

Chapter 2

PROJECT COORDINATION PROCESS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Table of Contents

List of Figures

CHAPTER 2

PROJECT COORDINATION PROCESS

2.1 INTRODUCTION

The Geotechnical Design Squads (GDSs) are located within the Regional Production Groups (RPGs). As indicated in Chapter 1, the RPGs consist of program management, road, bridge, hydraulic and utilities engineering in addition to the GDS. By placing the GDS with the other units within the RPG, project coordination is closer with an overall reduction in the lag time between project initiation and project completion and improves communication between the various design elements of a project.

2.2 PROJECT INITIATION

Geotechnical projects are initiated upon receipt of the request for surveys and subsurface utilities engineering (SUE). The request for surveys and SUE is typically received from either the Road Design Group or the Design Manager of the RPG. Upon receipt of the initiation documentation, the GDS will gather existing information from the project to include existing soils information, existing road and bridge plans and any preliminary plans depicting the proposed project. After collecting and reviewing this information, the GDS will schedule a Geoscoping trip to document site conditions and fill out a GDF 000 (see Appendix A) either during or immediately after the Geoscoping. During project initiation the Program Manager should provide information concerning whether a project will be Fast Track or Normal Track (see Figure 2-1). Fast Track projects will follow the coordination process depicted in Figure 2-2 and Normal Track projects will follow the coordination process depicted in Figure 2-3.

2.3 FAST TRACK PROJECTS

Fast Track projects are typically those projects that have limited or no environmental impacts, require no additional Right-of-Way, have relatively simple structures, and are placed on the same vertical and horizontal alignment as the existing bridge. These types of projects do not have surveys or hydraulic engineering analysis performed. Because survey data is typically not be available all references to depth should be from the existing bridge deck. Elevations are not be used in Fast Track projects. The geotechnical and structural designers are required to make a best estimate on the amount of scour anticipated at the bridge location. This estimate of scour should be based on the subsurface conditions encountered at the bridge site. Unlike a Normal Track, the preparation of a preliminary geotechnical exploration and report is not performed. Instead the GDS will issue a geotechnical advisory that contains the same information as the preliminary geotechnical report, except that the advisory will be based on available soils information from the general area, not the specific project location, unless available. Figure 2-2 provides the project coordination process that will be used for Fast Track projects. All borings should be performed within the existing SCDOT Right-of-Way and should not require the use of difficult access equipment to explore the site.

The GDS will receive layout plans from the Program Manager prior to commencing field work on the project. The Structures Design Group will provide anticipated loads for the proposed structure. The GDS will prepare a RGER and a BGER for the project. The reports will be provided to the respective Design Groups. Recommendations contained in the report will be incorporated into the project plans. In addition, the GDS will provide notes to be included on both the road and bridge plans. The GDS will review final bridge and road plans to assure that geotechnical design data were incorporated correctly in the plans. If required, the GDS will prepare Special Provisions in coordination with PCS/GDS.

2.4 NORMAL TRACK PROJECTS

As indicated above, the Program Manager will decide whether a project will be Normal or Fast Track. A Normal Track project will follow the coordination process depicted in Figure 2-3. Prior to initiating the preliminary geotechnical exploration, the GDS will compile available geotechnical information from the general area. The information should include, but not be limited to, existing subsurface explorations, pile load test data (static or dynamic) or pile installation records.

2.4.1 Preliminary Geotechnical Exploration

Upon completion of Geoscoping, the GDS will prepare a request for a Preliminary Geotechnical Investigation in accordance with the guidelines established in Chapter 4 of this Manual (see Figure 2-4). The request will be forwarded to the Geotechnical Materials Engineer of the Office of Materials and Research (OMR). The GDS will receive draft logs from OMR and will select samples for laboratory testing. After the completion of the laboratory testing, the GDS will receive the final soil test boring logs and laboratory testing results. Upon receipt of the final preliminary soil test boring records and laboratory work, the GDS shall prepare a Preliminary Geotechnical Report for both the bridge and road portions of the project. The reports shall be prepared in accordance with Chapter 21 of this Manual. The PGERs shall be forwarded to the appropriate Design Groups. In addition, the results of grain-size testing shall be forwarded to the Hydraulic Engineering Group for use in hydraulic design. The preliminary geotechnical exploration and preliminary geotechnical reports should be issued prior to the Design Field Review (DFR).

2.4.2 Right-of-Way Access Permission

Immediately prior to the DFR, the GDS will initiate the Right-of-Way (ROW) access permission process (see Figure 2-5), where permission will be obtained from adjacent landowners to access their property for the purpose of performing geotechnical explorations within the proposed new SCDOT Right-of-Way. If permission is obtained, then the GDS will prepare the final geotechnical exploration request and proceed as discussed below. If permission is denied, the GDS will develop a delay plan and discuss the plan with the Program Manager (see Figure 2-6). If the plan is acceptable, the GDS will continue into the final geotechnical exploration.

2.4.3 Final Geotechnical Exploration

After the completion of the DFR and receipt of the revised DFR plans, if required, the GDS will prepare a Final Geotechnical Investigation request in accordance with the guidelines established in Chapter 4 of this Manual. (See Figures 2-3 and 2-7). This request will be forwarded to OMR, Geotechnical Materials Engineer. The GDS will receive draft logs from OMR and will select samples for laboratory testing. After the completion of the laboratory testing, the GDS will receive the final soil test boring logs and laboratory testing results. The GDS will forward to the Hydraulic Engineering Group any additional subsurface information that has been collected during the final geotechnical exploration that may affect hydraulic design. The GDS will initiate the final geotechnical design upon receipt of the final soil boring logs. Figure 2-8 depicts the Final Geotechnical Design procedure.

The GDS will compile all geotechnical information for the project (existing, preliminary and final) for use in the final geotechnical design. The GDS will receive from the bridge and road squad final layouts for all structures and the bridge loading information. The GDS will prepare final bridge (BGER) and road (RGER) geotechnical reports in accordance with Chapter 21 of this Manual. In addition, the GDS will prepare Special Provisions that are required for the project. These Special Provisions will be prepared in coordination with the PCS/GDS. The GDS will review the final plans and specifications to assure that the geotechnical designs have been properly incorporated into the project design.

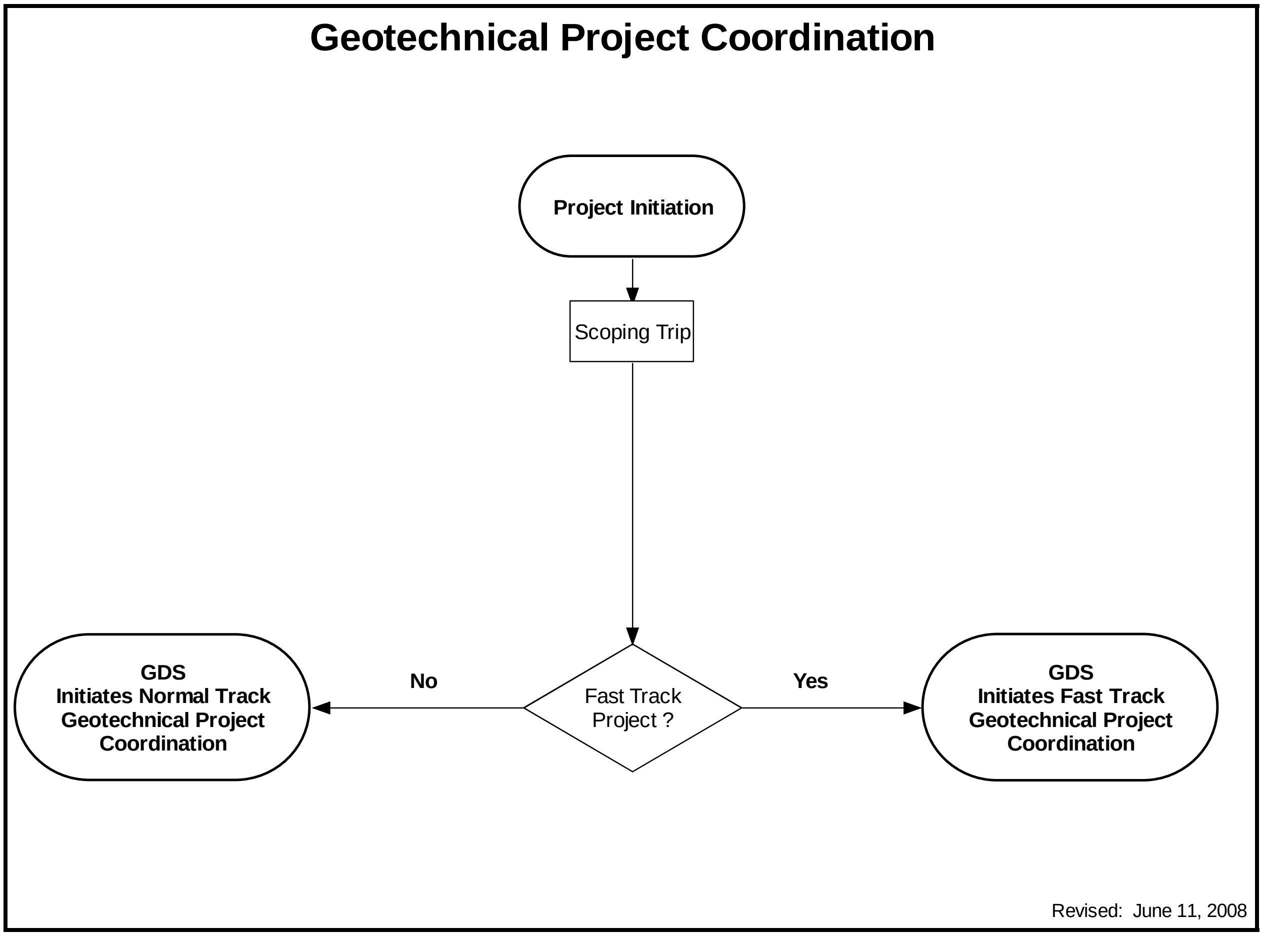


Figure 2-1, Project Initiation Process

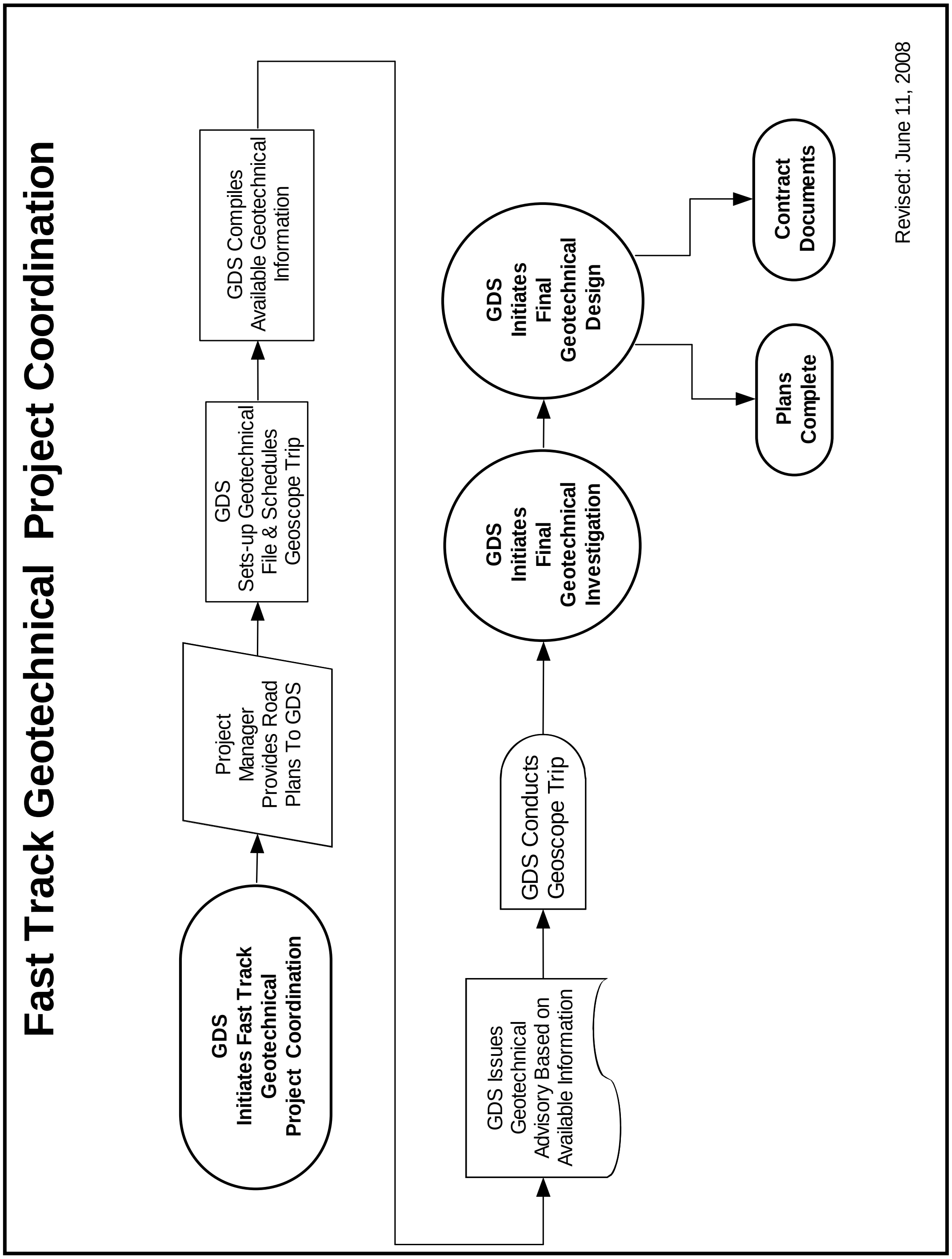


Figure 2-2, Fast Track Geotechnical Project Coordination

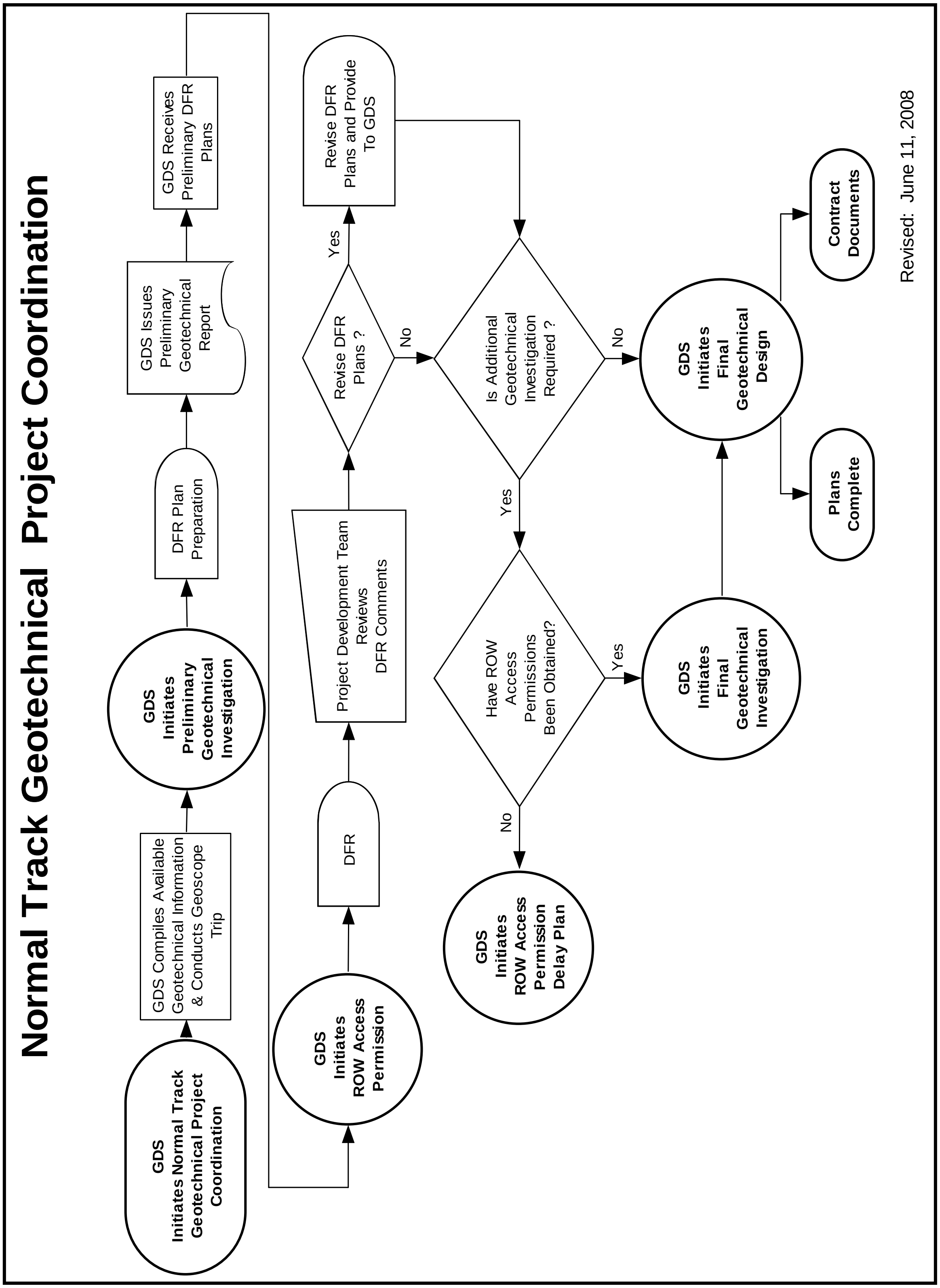


Figure 2-3, Normal Track Geotechnical Project Coordination

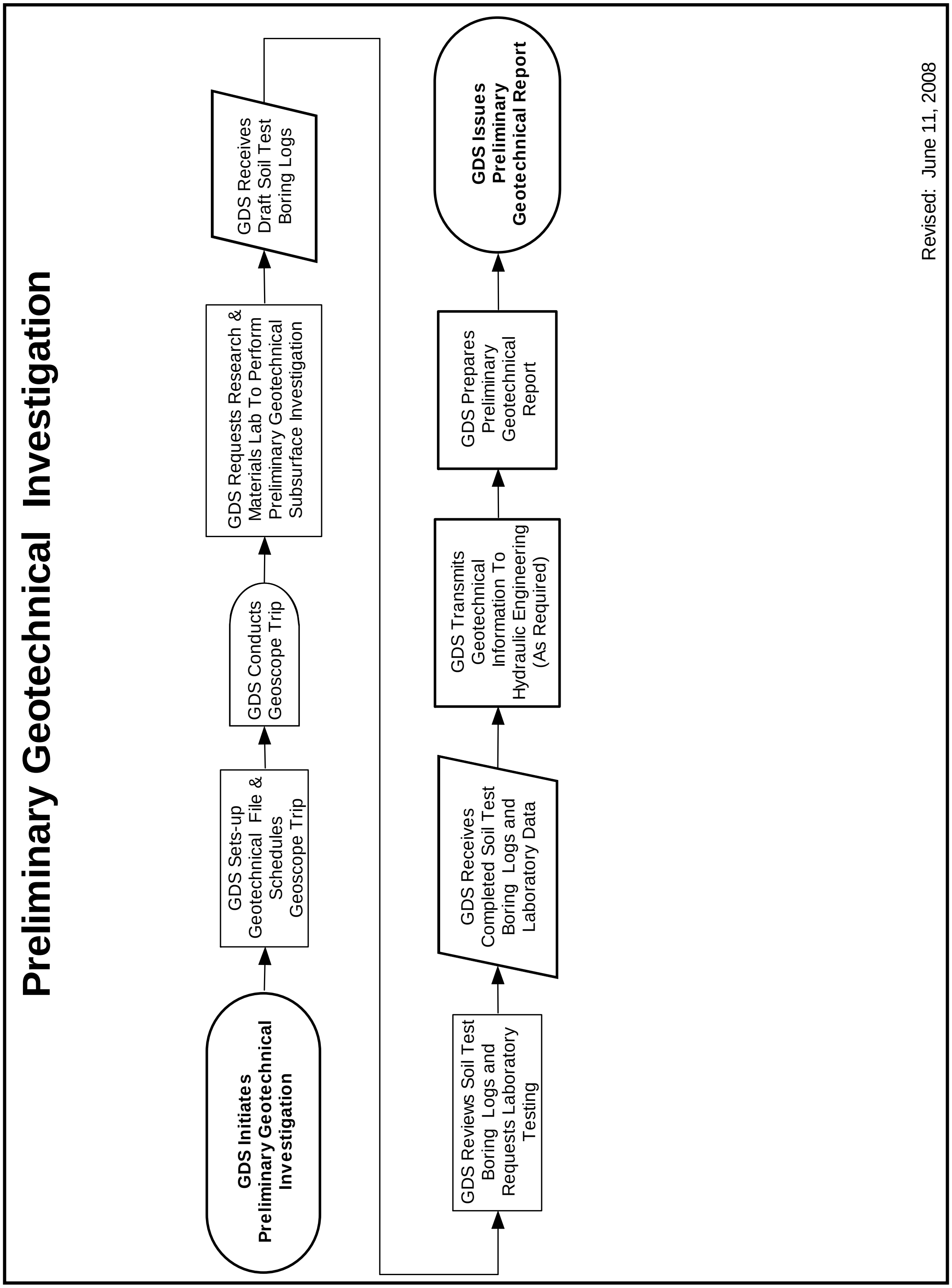


Figure 2-4, Preliminary Geotechnical Investigation

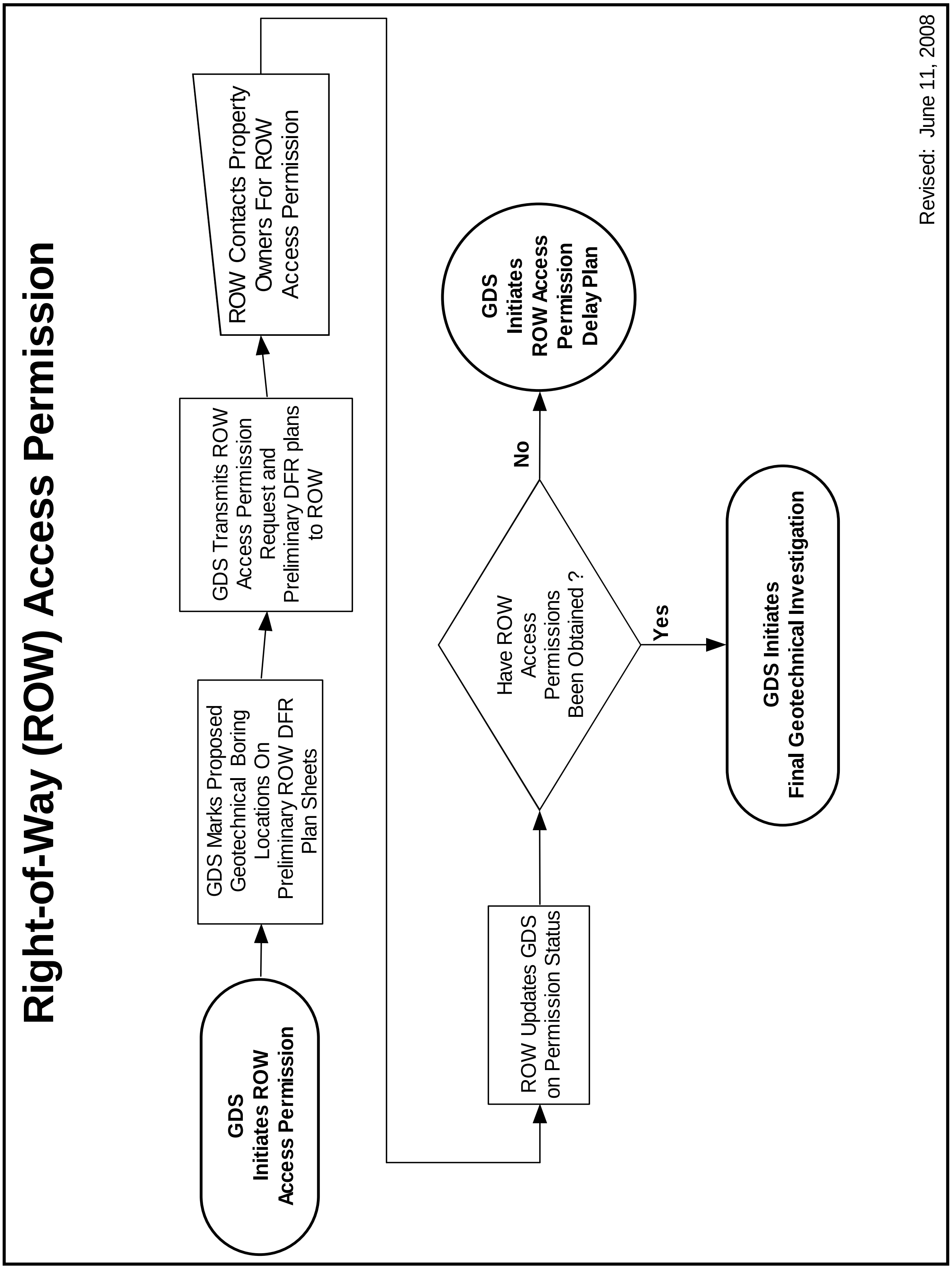


Figure 2-5, Right-of-Way Access Permission

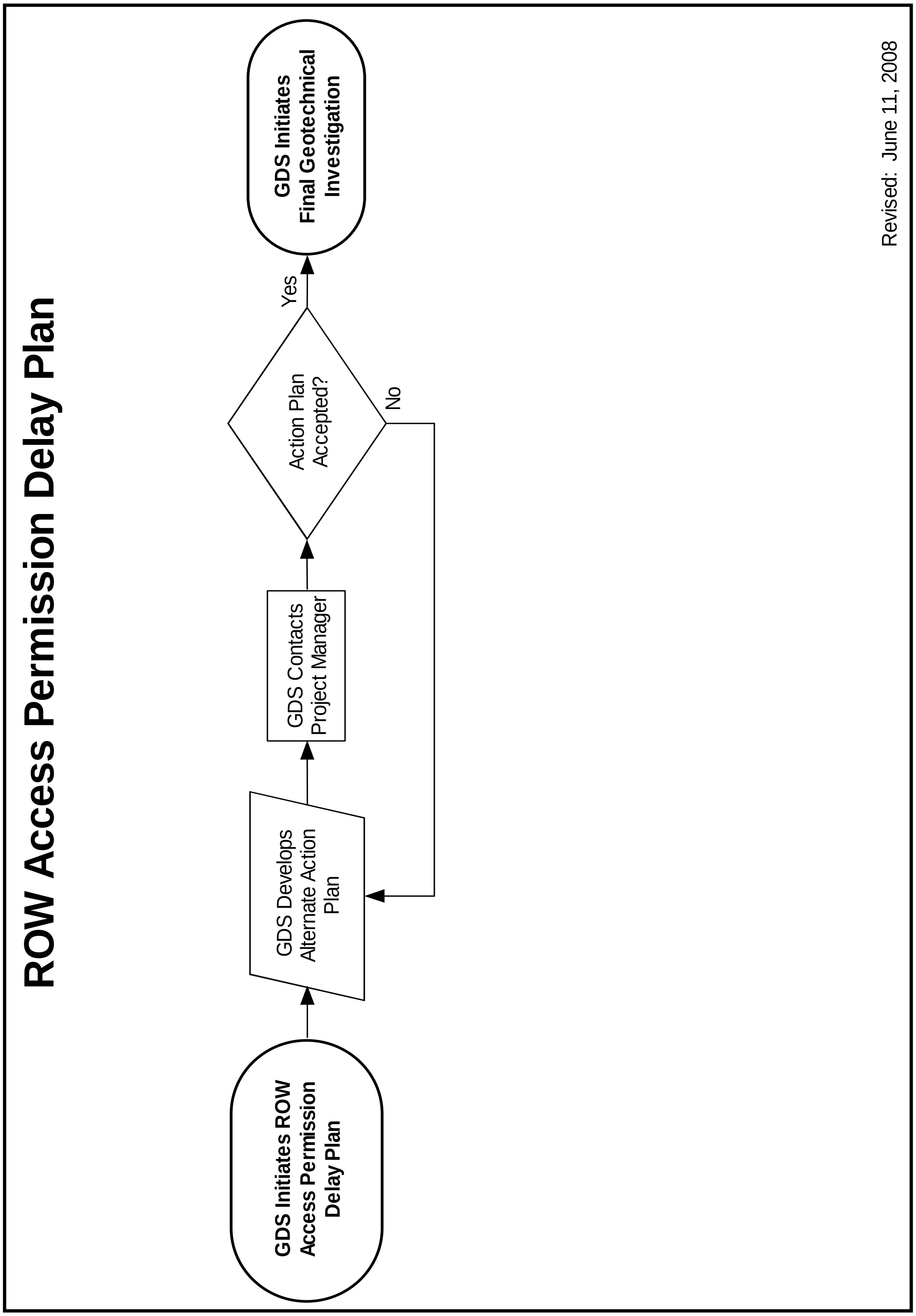


Figure 2-6, Right-of-Way Access Permission Delay Plan

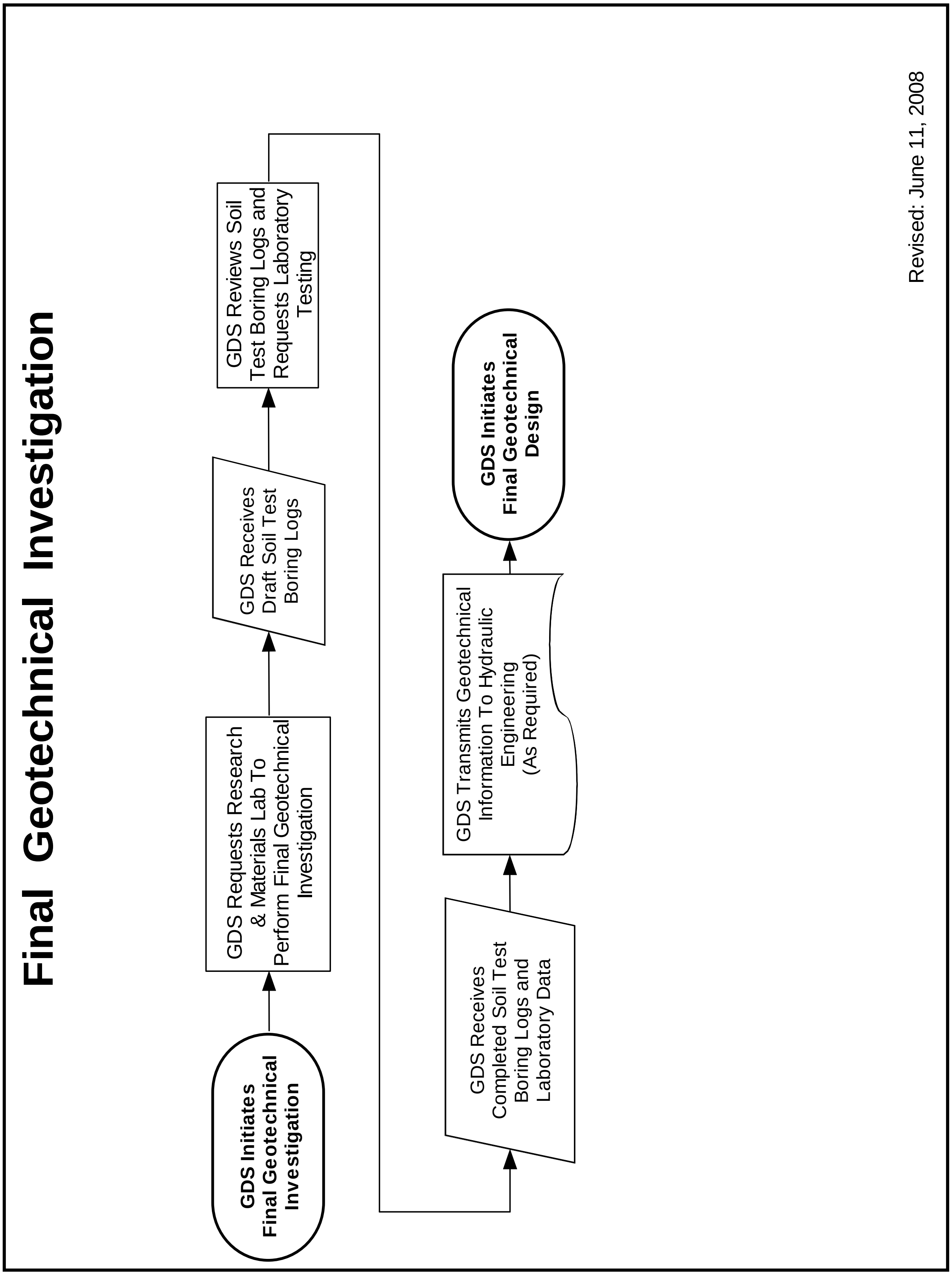


Figure 2-7, Final Geotechnical Investigation

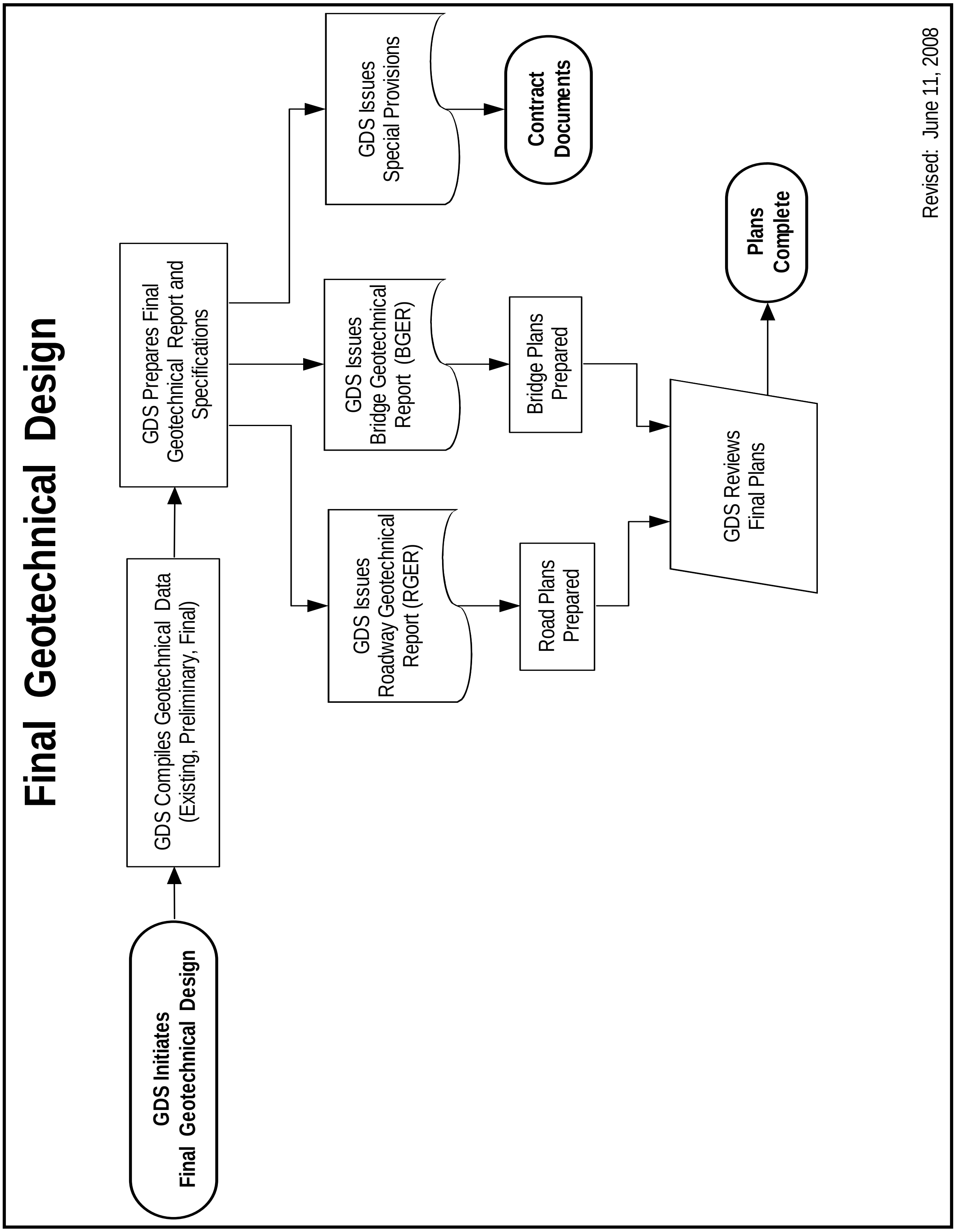


Figure 2-8, Final Geotechnical Design

Chapter 3

CONSULTANT SERVICES AND REVIEW

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Table of Contents

CHAPTER 3

CONSULTANT SERVICES AND REVIEW

3.1 INTRODUCTION

This Chapter presents the responsibilities of the Geotechnical Consultant to SCDOT. Consultants may be used in three different ways by SCDOT. First as a part of the on-call contract administered by SCDOT (either Geotechnical On-Call or General Services On-Call), secondly as part of a Traditional Design Team (i.e. design-bid-build) selected by SCDOT, and lastly as part of a Design-Build team. While the Geotechnical On-Call contract is used primarily for the subsurface exploration needs of the GDS, occasionally the Consultant may be requested to produce a full report, not just boring logs and laboratory services. The General Services On-Call is used primarily for providing Traditional Design Team Services and should be reviewed accordingly. There are times that the General Services On-Call may be used to provided drilling and laboratory services only. For those times, the General Services On-Call should follow the review process of the Geotechnical On-Call contract. Each use is discussed in greater detail in the following sections. In addition to the normal types of designs used, this Chapter will also discuss value engineering proposals and the review requirements for these proposals. The last section of this Chapter is concerned with the review of Consultant prepared geotechnical reports and the geotechnical elements of structural or roadway plans.

3.2 GEOTECHNICAL ON-CALL CONTRACT

The Geotechnical On-Call Contract is administered by the Office of Materials and Research (OMR). Consultants are selected for the contract based on the current SCDOT Procurement procedures. The primary purpose of the on-call contract is to provide subsurface exploration and laboratory testing services. The subsurface exploration and subsequent laboratory services will be conducted in accordance with the Request for Borings and Request for Laboratory Services prepared by the GDS. The GDS will prepare the subsurface exploration plan in accordance with Chapter 4 of this Manual and the requirements of the specific project. Laboratory testing requirements will be based on the needs of the specific project. All subsurface investigations and laboratory testing will be performed in general accordance with Chapters 5 and 6 of this Manual.

In addition to providing subsurface explorations, on occasion, the On-Call Consultant may be called upon to assist the GDS in the preparation of Geotechnical Reports. The engineering analysis and report will conform to the applicable Chapters in this Manual. The On-Call Consultant shall essentially function as an extension of the GDS.

3.3 TRADITIONAL DESIGN TEAM

The traditional design team concept is one where the design team is assembled and pursues a specific project as a team. The project is designed and plans are prepared for a letting to be bid by a Contractor. For these projects the Geotechnical Consultant will develop a subsurface exploration plan and submit the plan to the GDS for review and comment, prior to commencing field operations. The submitted plan will be reviewed by the GDS for compliance with the applicable Chapters of this Manual. The design team shall submit a preliminary geotechnical report along with preliminary construction plans for review by the GDS. The final geotechnical report shall be submitted along with the 95% bridge and road plans for review by the GDS. Comments made by the GDS on both the preliminary and final geotechnical reports shall be addressed to the satisfaction of the GDS. A corrected final copy of the geotechnical report shall be submitted to the GDS. In addition to the hard copy, the Geotechnical Consultant shall submit a .pdf file of the entire report including appendices to the GDS on a CD. The Geotechnical Consultant will comply with the appropriate Chapters of this Manual in preparing the subsurface exploration, engineering analysis and geotechnical reports.

3.4 DESIGN-BUILD

According to the AASHTO Joint Task Force on Design-Build, “Design-build is a project delivery method under which a project owner, having defined its initial expectations to a certain extent, executes a single contract for both architectural/engineering services and construction.” SCDOT has used the design-build process on several projects in recent years. The biggest difference between the Traditional Design Team and the Design-Build Team is that the construction typically commences while design is on-going. Typically SCODT will prepare “preliminary” plans for design-build projects. These plans consist of the approximate layout of the project (i.e. route and number of structures), the Design-Build Team is required to complete the final designs. As a part of this process, the GDS will issue a geotechnical base line report (see Chapter 21) indicating general geotechnical and geologic conditions. It is the responsibility of the Design-Build Team to identify geotechnical and geologic conditions that will impact the project and evaluate these impacts with regard to the designs being proposed.

The geotechnical design of design-build projects shall conform to the applicable sections of this Manual. Therefore, geotechnical reports are required to be submitted to the GDS for review. The Design-Build Team shall prepare a preliminary and final geotechnical report for all bridges, retaining walls, roadway embankments, concrete culverts and any other structures constructed for this type of project. The geotechnical report shall summarize subsurface soils, foundation design recommendations, laboratory testing results, soil test borings or in-situ testing logs, and locations of all soil investigations shown on the plans. Each report shall be submitted to the SCDOT along with the final or preliminary plan submittal. The review of the report will be performed in accordance with the structure submittal plan review process. Six (6) copies of each report shall be provided to the SCDOT prior to beginning foundation construction at each structure site. In addition, the Contractor shall provide a complete final copy of the report in .pdf format on a CD to the GDS. After construction of the foundations are complete, the Contractor shall provide a supplemental report containing the actual field conditions encountered, as-built foundation data and information, along with other geotechnical data collected during construction of the project.

3.5 VALUE ENGINEERING DESIGNS

The Department allows and encourages Contractors to submit Value Engineering Proposals. Upon receipt, the RCE will contact the appropriate SCDOT offices to discuss the original design intent and the potential merits and cost savings of accepting the Proposal. The GDS will be contacted for geotechnical items in the Proposal. More information on Value Engineering

Proposals may be found in the *SCDOT Standard Specifications*. All Value Engineering designs will be required to meet the criteria contained in the applicable chapters of this Manual.

3.6 REVIEW OF CONSULTANT SERVICES

Geotechnical reports will be reviewed by the GDS to ascertain that the reports comply with the applicable Chapters of this Manual. Deviations from the procedures outlined in this Manual should be indicated and should be explained as to why the procedures were changed. The report should also be read for clarity of ideas (i.e. is it easy to understand). Corrections to grammar or language should only be requested to clarify ideas (not wordsmithing). Any analysis provided should be reviewed for the reasonableness of the input parameters or assumptions used. It is not necessary to rerun the analysis unless a discrepancy is noted or suspected or if non-recognized software is used. For software not known to the GDS, additional information shall be requested on the software to include software designer and methods used to conduct the analysis. Examples of where the software has been used by another governmental agency or request side-by-side comparisons between recognized software and the non recognized software shall be required. It is the responsibility of the Geotechnical Consultant to verify that the software will achieve results similar to the software listed in Chapter 26 – Geotechnical Software.

3.7 REFERENCES

Current Design-Build Practices for Transportation Projects. AASHTO Joint Task Force on Design Build. January 2005.

South Carolina Department of Transportation, Bridge Design Manual, dated April 2006.

Chapter 4

SUBSURFACE INVESTIGATION GUIDELINES

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Table of Contents

List of Tables

CHAPTER 4

SUBSURFACE INVESTIGATION GUIDELINES

4.1 INTRODUCTION

A subsurface investigation is typically required for new or replaced structures, and roadway alignments involving earthwork. Examples of this include bridge replacements, widening of existing bridges and roadway realignments including widenings, retaining walls, box culverts, overhead sign-structures, sound barrier walls, and other miscellaneous structures.

This Chapter presents guidelines to be used in the development of subsurface investigations, for both preliminary and final. The actual type of investigation, depth, location, and frequency of all testing locations shall be based on project specific information. Subsurface investigations shall also indicate the testing intervals to be used if different from the standard intervals contained in this Chapter. The specific requirements for conducting field and laboratory testing are contained in Chapter 5 – Field and Laboratory Testing Procedures. The requirements of this Chapter shall be applied to in-house projects, projects designed by consultants, and design-build projects.

For projects designed by the RPGs, the subsurface investigation shall be prepared by the GDS prior to submission to the OMR or the RPG Design Manager for use in the General Services On-Call contract. The subsurface investigation plan shall also include all backup documentation used to develop the plan. This backup documentation includes, but is not limited to, previous soil borings in the general vicinity of the project, USDA soils maps, USGS topographic maps, aerial photographs, and wetland inventory maps. OMR is responsible for determining site accessibility and potential impacts to sensitive environmental areas. Site accessibility difficulties and impacts to sensitive environmental areas shall be discussed with the GDS prior to the relocation of any testing location. In addition, OMR is responsible for coordination of all traffic control issues for projects conducted under the Geotechnical On-Call contract.

For consultant projects, the Geotechnical Engineering Consultant shall submit to the GDS, for review and acceptance, a detailed subsurface investigation plan prior to the commencement of any field operations. The plan shall describe the soil or rock stratification anticipated as the basis of the planned exploration. The plan shall outline proposed testing types (borings/soundings), depths, and locations of all testing. The consultant's subsurface investigation plan shall conform to the requirements of this Manual. Frequently explorations must be conducted in sensitive environmental areas or in high hazard traffic areas, the consultant's exploration plan shall describe any special access requirements or traffic control requirements necessary to protect the interests of the Department during the field investigation phase. The Consultant is responsible for all special access requirements and traffic control. All traffic control shall conform to the latest Department guidelines.

4.2 SUBSURFACE INVESTIGATION

Subsurface investigations are typically conducted in two phases; preliminary and final. The location and spacing of all testing locations shall be coordinated between the preliminary and final subsurface investigations. The preliminary subsurface investigation should be conducted early enough in the design process to assist in the selection of foundation types and in determining the bridge/structure location and length and to identify areas requiring additional exploration during the final exploration. The testing locations for the preliminary subsurface investigation should be easily accessible and within the current Department Right-of-Way (ROW). The final subsurface investigation should account for the testing locations from the preliminary subsurface investigation. The requirements for the preliminary and final subsurface investigations are presented in the following sub-sections. The frequency and spacing of testing locations are presented in the following sections.

4.2.1 <u>Preliminary Subsurface Investigation</u>

The purpose of the preliminary subsurface investigation is to collect enough basic information to assist in development of preliminary plans. The contents of the Preliminary Geotechnical Engineering Report (PGER) for both bridge and road are presented in Chapter 21 - Geotechnical Reports. The testing locations should be located in readily accessible locations within the SCDOT ROW and should be, as indicated previously, coordinated with the final subsurface investigation. The preliminary subsurface investigation should include the collection of shear wave velocity data to depths of at least 100 feet from the existing ground surface. Shear wave velocity measurements may be extended to the practical limit of the equipment used to measure the shear wave velocities. These shear wave velocities will be used to determine the Site Class as described in Chapter 12 – Earthquake Engineering and the latest version of SCDOT *Seismic Design Specifications for Highway Bridges* including any addenda and/or amendments. The preliminary subsurface investigation will include a laboratory-testing program that will consist primarily of index testing. The laboratory-testing program shall also include grain-size analysis, including hydrometer, for soils within the upper 10 feet of the bottom of the water crossing. This analysis is required in determining the amount of scour predicted for the bridge over a body of water. The grain-size analysis shall be provided to the Hydraulic Engineering Group. Further electro-chemical testing shall be performed to determine the potential impacts of the soils, groundwater, and surface water on the structural components. In addition, a composite bulk sample shall be obtained of the existing embankment material. The composite sample shall have the following laboratory tests performed:

- Moisture Density Relationship (Standard Proctor)
- Grain Size Distribution with wash No. 200 Sieve
- Moisture-Plasticity Relationship Determination (Atterberg Limits)
- Natural Moisture Content
- Consolidation-Undrained Triaxial Shear Test with pore pressure measurements (sample remolded to 95 percent of Standard Proctor value)

The information (i.e. field and laboratory data) collected during the preliminary subsurface investigation will be used to refine the final subsurface investigation. The GDS, for in-house and Geotechnical Engineering Consultant for all other projects, is responsible for developing a soil exploration program and preparing the PGER. The bridge PGER provides general geotechnical recommendations based on limited soil information obtained from existing soil information and the preliminary subsurface investigation. The road PGER provides design recommendations for roadway earthwork and roadway structures. The general geotechnical recommendations are used to evaluate the DFR plans. After the DFR has been conducted, a detailed subsurface soil exploration is conducted based on the required structures defined during the DFR.

4.2.2 Final Subsurface Investigation

The purpose of the final subsurface investigation is to collect detailed subsurface information for use in developing geo-structural plans. The contents of the Bridge Geotechnical Engineering Report (BGER) and the Roadway Geotechnical Engineering Report (RGER) are presented in Chapter 21 - Geotechnical Reports. The testing locations shall be located along the proposed alignment of the roadway and bridge structure whether within or outside of the SCDOT ROW. The testing locations should be coordinated with the preliminary exploration to avoid testing in the same location and to assure that the entire construction area is adequately explored. The final subsurface investigation shall include a dilatometer sounding at each end bent. The information collected during the final subsurface investigation shall be used to develop the final foundation and earthwork recommendations for the project. The final subsurface investigation shall include any additional laboratory analyses. These additional laboratory analyses should include additional index property testing as well as sophisticated shear and consolidation testing.

4.3 SUBSURFACE INVESTIGATION METHODS

This section discusses the number, location and anticipated depth of all testing locations. As indicated previously, the preliminary and final subsurface investigations shall be coordinated to assure that the complete structure (whether bridge and roadway embankment) is adequately explored. The frequency and spacing of test locations will depend on the anticipated variation in subsurface conditions and the type of facility to be designed. A licensed surveyor shall locate (station, offset, and GPS coordinates (latitude and longitude)) and establish ground elevation at all soil test borings. The testing location frequency/spacing and depth criteria indicated below are the minimum requirements. Soil test borings (SPT borings), electro-piezocone (CPT) soundings and/or dilatometer (DMT) soundings are to be conducted at test locations. No more than half of the testing locations can be CPT or DMT soundings. DMT soundings should typically be limited to end bent areas only.

Soil test borings shall include the Standard Penetration Test (SPT). SPTs shall be conducted every 2 feet in the upper 10 feet of the subsurface (five samples) and every 5 feet below that depth. Since SPT samples are highly disturbed, these samples can only be used for index and classification testing. If high quality consolidation and shear strength data are required then undisturbed samples will be required. The collection of undisturbed samples (location and depth) shall be determined by the engineer-in-charge of the project. For projects located in the Lowcountry and Pee Dee Region (see Chapter 1) and for Aiken, Allendale, Bamberg, Barnwell, Calhoun, Lexington, Orangeburg and Richland Counties located in the Midland Regions, wash rotary drilling methods (see Chapter 5) shall be used. Variations to this requirement shall be made in writing and shall be forwarded to the PCS/GDS for review prior to approval.

In areas of difficult access beneath fill embankments, hand augers (HA) with dynamic cone penetrometers (DCPs) may be utilized to evaluate undercutting requirements. The DCPs should be performed approximately every foot.

4.3.1 Bridge Foundations

All bridges shall have soil testing taken at each end bent and at interior bents to meet the minimum geotechnical site investigation indicated below:

Table 4-1, Bridge Foundation Minimum Requirements

Bridge Foundation Type	Minimum Geotechnical Site Investigation
Pile Foundation	Minimum one testing location per bent[1]
Single Foundation - Drilled Shaft	Minimum one testing location per foundation location
Multiple Foundation – Drilled Shaft[2]	Minimum two testing locations per bent location
Shallow Foundation – Founded on Soil	Minimum three testing locations per bent location
Shallow Foundation – Founded on Rock	Minimum two testing locations per bent location

[1]Spacing between testing locations may be increased, but shall be approved prior to field operations and shall include justification, spacing may not exceed 100 feet.
[2]Minimum one testing location per bent in Lowcountry and Pee Dee Regions and in Aiken, Allendale, Bamberg, Barnwell, Calhoun, Lexington, Orangeburg and Richland Counties in Midlands Region.

All boring/soundings taken for deep foundations shall extend below the anticipated pile or drilled shaft tip elevation a minimum of 20 feet or a minimum of four times the minimum pile group dimension, whichever is deeper. All boring/soundings taken for shallow foundations shall extend beneath the anticipated bearing elevation as indicated in the following table:

Table 4-2, Minimum Depth of Investigation

Spread Footing Case	Minimum Testing Depth[1]
$L \leq 2B$	2B
$L \geq 5B$	4B
$2B \leq L \leq 5B$	3B

[1]Beneath the anticipated bearing elevation
L = Length of spread footing; B = Width of spread footing (minimum side dimension of footing)

All bridge foundations (deep and shallow) bearing on rock shall have a minimum of 20 feet of rock coring or the minimum testing depth requirements listed above, whichever is greater.

4.3.2 Retaining Walls

All retaining walls shall have one testing location performed at least every 75 feet along the wall line, if the wall is within 150 feet of bridge abutments. Retaining walls more than 150 feet from the bridge abutment shall have one testing location performed at least every 200 feet along the wall line. Anchored walls shall have testing locations at both the wall line and within the anchored zone at the same intervals specified above. The testing locations within the anchored zone shall be located approximately a distance equal to the height of the wall from the wall line. All testing locations shall be performed to a depth of at least twice the height of the wall beneath the anticipated bearing elevation or to auger refusal, whichever is shallower.

4.3.3 Embankments

All roadway embankments shall have one testing location performed at least every 500 feet along the roadway embankment. All testing locations shall be performed to a depth of at least twice the height of the embankment beneath the anticipated bearing elevation (i.e. to a depth sufficient to characterize settlement and stability issues) or to auger refusal, whichever is shallower.

4.3.4 Cut Excavations

All cut excavations shall have one test location performed at least every 300 feet along the cut area. All testing locations shall be performed to a depth of at least 25 feet below the anticipated bottom depth of the cut or to auger refusal, whichever is shallower. In addition, a composite bulk sample shall be collected from the area of the cut excavations. The composite sample shall have the following laboratory tests performed:

- Moisture Density Relationship (Standard Proctor)
- Grain Size Distribution with wash #200 Sieve
- Moisture-Plasticity Relationship Determination (Atterberg Limits)
- Natural Moisture Content
- Consolidation-Undrained Triaxial Shear Test with pore pressure measurements (sample remolded to 95% of Standard Proctor value)

4.3.5 Culverts

All new crossline culverts (pipe, box, or floorless) shall have a minimum of one test location at each end of the culvert and at every 100 feet of the new crossline culvert. Crossline culvert extensions shall have a minimum of one test location at each extension. For crossline culvert extensions greater than 50 feet, testing locations shall be spaced every 50 feet. All testing locations shall extend to a depth beneath the anticipated bearing elevation of at least twice the height of the embankment or in accordance with the bridge spread footing criteria, whichever is deeper. Testing may be terminated above these depths if auger refusal is encountered.

4.3.6 Sound Barrier Walls

Sound barrier walls may be supported by either shallow foundations or deep foundations depending on the foundation system selected by the contractor. For sound barrier walls located on top of a berm, the testing locations shall extend a minimum of twice the berm height plus twice the height of the proposed sound barrier wall for shallow foundations. For sound barrier walls not located on top of a berm, the testing locations shall extend a minimum of twice the height of the proposed sound barrier wall for shallow foundations. If deep foundations are used to support the sound barrier walls, the testing shall extend a minimum of 20 feet beneath the anticipated deep foundation tip elevation. Testing locations for sound barrier walls shall be placed at the beginning and ending of the wall, at the location of major changes in the wall alignment and at a minimum spacing of 200 feet between these locations.

4.3.7 Miscellaneous Structures

Miscellaneous structures such as overhead signs and light poles shall have a minimum of one test location performed per foundation location unless directed otherwise by the PCS/GDS. All test locations shall extend to the same depth criteria as specified for the bridge test locations for the same type of foundation.

4.3.8 Pavement Structures

Subsurface investigation requirements for pavement structure design vary with location, traffic level, and project size. Requirements for pavement structure design subsurface investigations are provided in SCDOT's *Pavement Design Guidelines* (latest edition), which is published by the OMR. Contact the OMR Geotechnical Materials Engineer for further information.

Chapter 5

FIELD AND LABORATORY TESTING PROCEDURES

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Table of Contents

List of Tables

CHAPTER 5

FIELD AND LABORATORY TESTING PROCEDURES

5.1 INTRODUCTION

This Chapter discusses items related to field and laboratory testing procedures. The first item is sampling procedures and will discuss the different methods of retrieving soil and rock samples. The second item is the drilling procedure and discusses what types of equipment are typically available. The third item is the soil/rock laboratory testing and will discuss the different types of testing procedures. Tests shall be performed in accordance with ASTM and/or AASHTO.

5.2 SAMPLING PROCEDURE

5.2.1 Soil Sampling

ASTM and AASHTO have procedures that must be followed for the collection of field samples. All samples must be properly obtained, preserved, and transported to a laboratory facility in accordance with these procedures in order to preserve the samples as best as possible. There are several procedures that can be used for the collection of samples as described below. See ASTM D4220 - *Standard Practices for Preserving and Transporting Soil*.

5.2.1.1 Bulk Samples

Bulk samples are highly disturbed samples obtained from auger cuttings or test pits. The quantity of the sample depends on the type of testing to be performed, but can range up to 50 lb (25 kg) or more. Typical testing performed on bulk samples include moisture-density relationship, moisture-plasticity relationship, grain-size distribution, natural moisture content, and triaxial compression on remodeled specimens.

5.2.1.2 Split-Barrel Sampling

The most commonly used sampling method is the split-barrel sampler, also known as standard split-spoon. This method is used in conjunction with the Standard Penetration Test. The sampler is driven into soil by means of hammer blows. The number of blows required for driving the sampler through three 6-inch intervals is recorded. The last two 6-inch intervals is added to make up the standard penetration number, N_{meas}. After driving is completed the sampler is retrieved and the soil sample is removed and placed into air tight containers. Each standard penetration number and collection of samples is to be done at 5-foot intervals, except in the upper 10 feet where samples will be collected every 2 feet. This type of sampling is adequate for moisture content, grain-size distribution, Atterberg Limits tests, and visual identification. See ASTM D1586 - *Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils* (AASHTO T206 - *Standard Method of Test for Penetration Test and Split-Barrel Sampling of Soil*s).

5.2.1.3 Shelby Tube

The Shelby tube is a thin-walled steel tube pushed into the soil to be sampled by hydraulic pressure and spun to shear off the base. Afterwards the sampler is pulled out and immediately sealed and taken to the laboratory facility. This process allows the sample to be undisturbed as much as possible and is suitable for fine-grained soils that require strength and consolidation tests. See ASTM D1587 - *Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes* (AASHTO T207 - *Standard Method of Test for Thin-Walled Tube Sampling of Soils*). There are a variety of methods that may be used to collect a Shelby tube samples. Listed in the following sections are the types of sampling methods commonly used. It is not the intention of this Manual that this list be comprehensive. If another sampling procedure/method is to be used, contact the PCS/GDS for review prior to acceptance.

5.2.1.3.1 Fixed Piston Sampler

This sampler has the same standard dimensions as the Shelby tube, above. A piston is positioned at the bottom of the thin-wall tube while the sampler is lowered to the bottom of the hole, thus preventing disturbed materials from entering the tube. The piston is locked in place on top of the soil to be sampled. A sample is obtained by pressing the tube into the soil with a continuous, steady thrust. The stationary piston is held fixed on top of the soil while the sampling tube is advanced. This creates suction while the sampling tube is retrieved thus aiding in retention of the sample. This sampler is suitable for soft to firm clays and silts. Samples are generally less disturbed and have a better recovery ratio than those from the Shelby tube method.

5.2.1.3.2 Floating Piston Sampler

This sampler is similar to the fixed method above, except that the piston is not fixed in position but is free to ride on the top of the sample. The soils being sampled must have adequate strength to cause the piston to remain at a fixed depth as the sampling tube is pushed downward. If the soil is too weak, the piston will tend to move downward with the tube and a sample will not be obtained. This method should therefore be limited to stiff or hard cohesive materials.

5.2.1.3.3 Retractable Piston Sampler

This sampler is similar to the fixed piston sampler; however, after lowering the sampler into position the piston is retracted and locked in place at the top of the sampling tube. A sample is then obtained by pushing the entire assembly downward. This sampler is used for loose or soft soils.

5.2.1.3.4 Hydraulic (Osterberg) Piston Sampler

The hydraulic piston sampler is made similar to the Shelby tube. Instead of a rod pushing the sampler into the soil and then spun to shear off, the thin walled tube is pushed into the soil and a piston closes the end of the thin walled tube. After the tube closes, pressure is released thus preventing distortion by neither letting the soil squeeze into the sampler tube very fast nor admitting excess soil. This technique is especially useful for soil samples that require the most undisturbed sample in soft clays and silts.

5.2.2 Rock Core Sampling

The most common method for obtaining rock samples is diamond core drilling. There are three basic types of core barrels: Single tube, double tube, and triple tube. See ASTM D2113 - *Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation* (AASHTO T225 - *Standard Method of Test for Diamond Core Drilling for Site Investigation*).

5.3 FIELD TESTING PROCEDURES

Assuming access and utility clearances have been obtained and a survey base line has been established in the field, field explorations are begun based on the subsurface exploration request prepared by the GDS for in-house or by the Geotechnical Engineering Consultant for all other projects. Many methods of field exploration exist; some of the more common are described below. These methods are often augmented by in-situ testing. The testing described in this Chapter provides the Geotechnical Engineer with soil and rock parameters determined in-situ. This is important on all projects, especially those involving soft clays, loose sands, and/or sands below the water table, due to the difficulty of obtaining representative samples suitable for laboratory testing. For each test included, a brief description of the equipment, the test method, and the use of the data is presented.

5.3.1 Test Pits

These are the simplest methods of inspecting subsurface soils. Test pits consist of excavations performed by hand, backhoe, or dozer. Hand excavations are often performed with posthole diggers. Test pits offer the advantages of speed and ready access for sampling; however, test pits are severely hampered by limitations of depth and by the fact that advancement through soft or loose soils or below the water table can be extremely difficult. Test pits are used to examine large volumes of near surface soils and can be used to obtain bulk samples for additional testing.

5.3.2 Soil Borings

Soil borings are probably the most common method of exploration. Soil borings can be advanced using a number of methods. In addition, several different in-situ tests can be performed in the open borehole. The methods for advancing the boreholes will be discussed first followed by the methods of in-situ testing.

5.3.2.1 Manual Auger Borings

Manual auger borings are advanced using hand held equipment. Typically, these borings are conducted in areas where access for standard drilling equipment is severely restricted. Manual auger borings are limited in depth by the presence of ground water or collapsible soils that cause caving in the borehole. The Dynamic Cone Penetrometer test is usually conducted in conjunction with this boring method.

5.3.2.2 Hollow Stem Auger Borings

A hollow-stem auger (HSA) consists of a continuous flight auger surrounding a hollow drill stem. The hollow-stem auger is advanced similar to other augers; however, removal of the hollow stem auger is not necessary for sampling. SPT and undisturbed samples are obtained through the hollow drill stem, which acts like a casing to hold the hole open. This increases usage of hollow-stem augers in soft and loose soils. See ASTM D6151 - *Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling* (AASHTO T306 - *Standard Method of Test for Progressing Auger Borings for Geotechnical Explorations*). This drilling method is limited to areas where the ground water is not anticipated effecting the Standard Penetration Test (SPT).

5.3.2.3 Wash Rotary Borings

In this method, the boring is advanced by a combination of the chopping action of a light bit and the jetting action of water flowing through the bit. A downward pressure applied during rapid rotation advances the hollow drill rods with a cutting bit attached to the bottom. The drill bit cuts the material and drilling fluid washes the cuttings from the borehole. This is, in most cases, the fastest method of advancing the borehole and can be used in any type of soil except those containing considerable amounts of large gravel or boulders. Drilling mud or casing can be used to keep the borehole open in soft or loose soils, although the former makes identifying strata change by examining the cuttings difficult. SPT and undisturbed samples are obtained through the drilling fluid, which holds the borehole open. This method of drilling is required in the Lowcountry and the Pee Dee Regions and in Aiken, Allendale, Bamberg, Barnwell, Calhoun, Lexington, Orangeburg and Richland Counties of the Midlands Region (see Chapter 1).

5.3.2.4 Coring

A core barrel is advanced through rock by the application of downward pressure during rotation. Circulating water removes ground-up material from the hole while also cooling the bit. The rate of advance is controlled so as to obtain the maximum possible core recovery. See ASTM D2113 – *Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation*. A professional geologist or geotechnical engineer shall be on-site during coring operations to perform measurements in the core hole to allow for determination of the Rock Mass Rating (RMR) (see Chapter 6).

5.3.3 Standard Penetration Test

This test is probably the most widely used field test in the United States. It has the advantages of simplicity, the availability of a wide variety of correlations for its data, and the fact that a sample is obtainable with each test. A standard split-barrel sampler (discussed previously) is advanced into the soil by dropping a 140-pound safety or automatic hammer attached to the drill rod from a height of 30 inches. **[Note: Use of a donut hammer is not permitted]**. The sampler is advanced a total of 18 inches. The number of blows required to advance the sampler for each of three 6-inch increments is recorded. The sum of the number of blows for the second and third increments is called the Standard Penetration Value, or more commonly, N-value (N_{meas}) (blows per foot). Tests shall be performed in accordance with ASTM D1586 - *Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils* (AASHTO T206 - *Standard Method of Test for Penetration Test and Split-Barrel Sampling of Soils*). The

Standard Penetration Test shall be performed every 2 feet in the upper 10 feet (5 N_{meas}) and every 5 feet thereafter. The exception is beneath embankments, the Standard Penetration Test will also be performed every 2 feet in the first 10 feet of original ground surface. The depth to the original ground surface may be estimated based on the height of the existing embankment.

When Standard Penetration Tests (SPT) are performed in soil layers containing shell or similar materials, the sampler may become plugged. A plugged sampler will cause the SPT N-value to be much larger than for an unplugged sampler and, therefore, not a representative index of the soil layer properties. In this circumstance, a realistic design requires reducing the N-value used for design to the trend of the N-values which do not appear distorted. However, the actual N-values should be presented on the Soil Test Boring Logs (see Chapter 6). A note shall be placed on the Soil Test Boring Logs indicating that the sampler was plugged.

The SPT values should not be used indiscriminately. They are sensitive to the fluctuations in individual drilling practices and equipment. Studies have also indicated that the results are more reliable in sands than clays. Although extensive use of this test in subsurface exploration is recommended, it should always be augmented by other field and laboratory tests, particularly when dealing with clays. The type of hammer (safety or automatic) shall be noted on the boring logs, since this will affect the actual input driving energy. N_{meas} require correction prior to being used in engineering analysis (see Chapter 7).

The amount of driving energy shall be measured using ASTM D4633 - *Standard Test Method for Energy Measurement for Dynamic Penetrometers*. Since there is a wide variability of performance in SPT hammers, this method is used to evaluate an individual hammer's performance. The energy of a hammer can be effected by the mechanical state of the hammer system (i.e. maintained or not), the condition of the rope, the experience of the driller, the time of day, and the weather. For SPTs performed under the Geotechnical On-Call Contract, a QA/QC plan is required. For SPTs performed under the General Services On-Call Contract, a QA/QC plan for measuring hammer energy is also required. The QA/QC plans under either contract shall be submitted to the Department for acceptance, prior to being used in the field.

The SPT installation procedure is similar to pile driving because it is governed by stress wave propagation. As a result, if force and velocity measurements are obtained during a test, the energy transmitted can be determined.

5.3.4 Dynamic Cone Penetrometer

The Dynamic Cone Penetrometer is a dynamic penetration test usually performed in conjunction with manual auger borings. Dynamic Cone Penetrometer testing shall be conducted using the procedure presented by Sowers and Hedges (1966). The Dynamic Cone Penetrometer resistance values shall be correlated to N_{meas}, by performing an SPT adjacent to the Dynamic Cone Penetrometer test.

5.3.5 Cone Penetrometer Test

The Cone Penetrometer Test is a quasi-static penetration test in which a cylindrical rod with a conical point is advanced through the soil at a constant rate and the resistance to penetration is measured. A series of tests performed at varying depths at one location is commonly called a sounding.

Several types of penetrometer are in use, including mechanical (Dutch) cone, mechanical friction-cone, electric cone, electric friction-cone, and electro-piezocone. Cone penetrometers measure the resistance to penetration at the tip of the penetrometer or end-bearing component of resistance. Friction-cone penetrometers are equipped with a friction sleeve, which provides the added capability of measuring the side friction component of resistance. Mechanical penetrometers have telescoping tips allowing measurements to be taken incrementally, generally at intervals of 8 inches (200 mm) or less. Electronic penetrometers use electronic force transducers to obtain continuous measurements with depth. Electro-piezocones are also capable of measuring pore water pressures during penetration. Electro-piezocones or some variation (i.e. seismic electro-piezocones) are the only allowed cone penetrometed device.

For all types of penetrometers, cone dimensions of a 60-degree tip angle and a 10 cm^2 (1.55 in^2) projected end area are standard. Friction sleeve outside diameter is the same as the base of the cone. Penetration rates should be between 10 to 20 mm/sec. Tests shall be performed in accordance with ASTM D5778 - *Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils* (electro-piezocones).

The penetrometer data is plotted showing the tip stress, the friction resistance and the friction ratio (friction resistance divided by tip stress) vs. depth. Pore pressures, can also be plotted with depth. The results should also be presented in tabular form indicating the interpreted results of the raw data. See Chapter 6 – Materials Description, Classification and Logging for presentation of CPT data.

The friction ratio plot can be analyzed to determine soil type. Many correlations of the cone test results to other soil parameters have been made, and design methods are available for spread footings and piles. The penetrometer can be used in sands or clays, but not in rock or other extremely dense soils. Generally, soil samples are not obtained with soundings, so penetrometer exploration should always be augmented by SPT borings or other borings with soil samples taken. Since soil samples are not obtained, the CPT should be correlated to the in-situ soils by performing a boring adjacent to the sounding.

The electro-piezocones can also be used to measure the dissipation rate of the excessive pore water pressure. This type of test is useful for subsoils, such as fibrous peat, muck, or soft clays that are very sensitive to sampling techniques. The cone should be equipped with a pressure transducer that is capable of measuring the induced water pressure. To perform this test, the cone will be advanced into the subsoil at a standard rate of 20 mm/sec. Pore water pressures will be measured immediately and at several time intervals thereafter. Use the recorded data to plot pore pressure dissipation versus log-time graph. Using this graph, direct calculation of the pore water pressure dissipation rate or rate of settlement of the soil can be performed.

Electro-piezocones can be fitted with other instrumentation above the friction sleeve. The additional instrumentation can include geophones that may be used to measure shear wave velocities. Another instrument that may be included is an inclinometer to determine if the instrument is getting off plumb. Other instruments include microphones and nuclear density equipment.

5.3.6 Dilatometer Test

The dilatometer is a 3.75-inch wide and 0.55-inch thick stainless steel blade with a thin 2.4-inch diameter expandable metal membrane on one side. While the membrane is flush with the blade surface, the blade is either pushed or driven into the soil using a drilling rig. Rods carry pneumatic and electrical lines from the membrane to the surface. At depth intervals of 12 inches, pressurized gas is used to expand the membrane, both the pressure required to begin membrane movement and that required to expand the membrane into the soil 0.04 inches (1.1 mm) are measured. Additionally, upon venting the pressure corresponding to the return of the membrane to its original position may be recorded. Through developed correlations, information can be deduced concerning material type, pore water pressure, in-situ horizontal and vertical stresses, void ratio or relative density, modulus, shear strength parameters, and consolidation parameters. Compared to the pressuremeter, the flat dilatometer has the advantage of reduced soil disturbance during penetration. Tests shall be performed in accordance with ASTM D6635 - *Standard Test Method for Performing the Flat Plate Dilatometer*.

5.3.7 Pressuremeter Test

This test is performed with a cylindrical probe placed at the desired depth in a borehole. The Menard type pressuremeter requires pre-drilling of the borehole; the self-boring type pressuremeter advances the hole itself, thus reducing soil disturbance. The PENCEL pressuremeter can be set in place by pressing it to the test depth or by direct driving from ground surface or from within a predrilled borehole. The hollow center PENCEL probe can be used in series with the static cone penetrometer. The Menard probe contains three flexible rubber membranes. The middle membrane provides measurements, while the outer two are “guard cells” to reduce the influence of end effects on the measurements. When in place, the guard cell membranes are inflated by pressurized gas while the middle membrane is inflated with water by means of pressurized gas. The pressure in all the cells is incremented and decremented by the same amount. The measured volume change of the middle membrane is plotted against applied pressure. Tests shall be performed in accordance with ASTM D4719 - *Standard Test Method for Prebored Pressuremeter Testing in Soils.*

Studies have shown that the “guard cells” can be eliminated without sacrificing the accuracy of the test data provided the probe is sufficiently long. Furthermore, pumped air can be substituted for the pressurized gas used to inflate the membrane with water. The TEXAM® pressuremeter is an example of this type.

Results are interpreted based on semi-empirical correlations from past tests and observation. In-situ horizontal stresses, shear strength, bearing capacities, and settlement can be estimated using these correlations. The pressuremeter test results can be used to obtain load transfer curves (p-y curves) for lateral load analyses. The pressuremeter test is very sensitive to borehole disturbance and the data may be difficult to interpret for some soils.

5.3.8 Field Vane Test

This test consists of advancing a four-bladed vane into cohesive soil to the desired depth and applying a measured torque at a constant rate until the soil fails in shear along a cylindrical surface. The torque measured at failure provides the undrained shear strength of the soil. A second test run immediately after remolding at the same depth provides the remolded strength of the soil and thus information on soil sensitivity. Tests shall be performed in accordance with ASTM D2573 - *Standard Test Method for Field Vane Shear Test in Cohesive Soil* (AASHTO T223 - *Standard Method of Test for Field Vane Shear Test in Cohesive Soil*).

This method is commonly used for measuring shear strength in soft clays and organic deposits. It should not be used in stiff and hard clays. Results can be affected by the presence of gravel, shells, roots, or sand layers. Shear strength may be overestimated in highly plastic clays and a correction factor should be applied.

5.3.9 <u>Geophysical Testing Methods</u>

Geophysical testing methods are non-destructive testing procedures which can provide general information on the general subsurface profile, depth to bedrock or water, location of granular borrow areas, peat deposits or subsurface anomalies and provide an indication of certain material properties (i.e. compression wave (V_P) and shear wave velocity (V_S)). Geophysical testing methods are not limited to subsurface conditions, but can also be used to evaluate existing bridge decks, foundations and pavements. The reader should see Application of Geophysical Methods to Highway Related Problems, FHWA-IF-04-021, for additional information on the application of geophysical test methods to other areas other than subsurface conditions.

5.3.9.1 Surface Wave Methods

Surface wave methods consist of Spectral Analysis of Surface Waves (SASW) or Multi-channel Analysis of Surface Waves (MASW). The SASW and MASW are used to measure layer thickness, depth and the shear wave velocity (V_S) of the layer. The shear wave velocity is more of bulk (general) velocity than a discrete velocity of a layer. Discrete shear wave velocity may be determined by crosshole or downhole methods. While the SASW will typically have 2 geophones, the MASW will have additional geophones spread over a larger area. Typically SASW and the MASW profiles are limited to a depth of approximately 130 feet using man portable equipment. Additional depth can be obtained but heavier motorized equipment is required.

5.3.9.2 Downhole Shear Wave Velocity Methods

Downhole methods for determining shear wave velocity differ from surface methods in that equipment is placed in the ground. In downhole methods, either a casing is placed in the ground and geophone is lowered in the casing or a seismic cone penetrometer (SCPT) is pushed into the ground. The SCPT has a geophone typically mounted above the friction sleeve on the cone. With either method, a shear wave is induced at the ground surface and the time for arrival is determined. If casing is used, care must be taken during construction. One of the major limitations of the SCPT is refusal to advance in dense soils.

5.3.9.3 Crosshole Shear Wave Velocity Methods

In crosshole shear wave velocity testing, shear wave velocities are determined between a series of casings. A downhole hammer and geophone are lowered to the same depth, but in different holes. The hammer is tripped and time for the shear wave to travel to the geophone is recorded. The major limitation to the crosshole method is the expense of the installation of the required casings. In addition, the care that must be taken during the construction of the casings to assure that the casings are plumb and in the same horizontal plane.

5.3.9.4 Seismic Refraction

Seismic refraction is used to determine the depth to bedrock. This method works well for depths less than 100 feet. A seismic energy source is required for producing seismic waves. A sledge hammer is typically used for depths less than 50 feet and either a drop weight or a black powder charge is used for depths between 50 and 100 feet. The seismic compression waves penetrate the overburden material and refract along the bedrock surface. This method can be used for up to 4 soil on rock layers; however, each layer must have a higher shear wave velocity than the overlying layer.

5.3.9.5 Seismic Reflection

Seismic reflection uses a surface seismic wave source to create seismic waves that can penetrate the subsurface. The waves are reflected at interfaces that have either a change in shear wave velocity and/or a change in density. Changes in velocity or density are termed impedance contrasts. At impedance contrasts, a portion of the seismic wave is reflected back to the ground surface and a portion continues into the subsurface where it is reflected at the next impedance contrast. Seismic reflection techniques can obtain information in excess of 100 feet.

5.3.9.6 Resistivity

Resistivity is used to find the depth to bedrock since soil and rock typically have different electrical resistances. The depth of the resistivity survey is typically 1/3 of the electrode spacing. For example, to reach a depth of 50 feet an electrode spacing of 150 feet is required. Resistivity surveys can reach depths of 160 feet. Resistivity testing is affected by the moisture content of the soil and the presence or lack of metals, salts and clay particles. In addition, resistivity surveys may be used to model ground water flow through the subsurface. Further, resistivity surveys may also be used to determine the potential for corrosion of foundation materials for the in-situ subsurface materials.

5.4 SOIL/ROCK LABORATORY TESTING

5.4.1 <u>Grain-Size Analysis</u>

There are two types of tests: Grain-Size with wash No. 200 and Hydrometer test. Grain size with wash No. 200, also known as Sieve Analysis, is for coarse-grained soils (sand, gravels). The hydrometer analysis is used for fine-grained soils (clays, silts).

The results of the analyses are presented in a semilogarithmic plot known as particle-size distribution curves. In the semilogarithmic scale, the particle sizes are plotted on the log scale. The percent finer is plotted in arithmetic scale. Therefore, the graph is easy to read the percentages of gravel, sand, silt, and clay-size particles in a sample of soil.

The grain-size analysis can also be used for obtaining three basic soil parameters from the curves. These parameters are: effective size (D10), Coefficient of Uniformity (Cu), and Coefficient of Curvature (Cc). The Hydraulic Engineering Group requires these parameters for scour analysis. Those soil test-boring logs at the Interior Bents of a bridge over a water environment must have a Hydrometer test performed at depths from 0-5 ft. See ASTM D422 - *Standard Test Method for Particle-Size Analysis of Soils* (AASHTO T88 - *Standard Method of Test for Particle Size Analysis of Soils*).

5.4.1.1 Sieve Analysis

The sieve analysis is a method used to determine the grain size distribution of soils. The soil is passed through a series of woven wires with square openings of decreasing sizes. The test gives a soil classification based on the percentage retained on the sieve. See ASTM C136 - *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates* (AASHTO T311 - *Standard Method of Test for Grain-Size Analysis of Granular Soil Materials*).

5.4.1.2 Hydrometer

The Hydrometer analysis is used to determine the particle size distribution in a soil that is finer than a No. 200 sieve size (0.075 mm), which is the smallest standard size opening in the sieve analysis. The procedure is based on the sedimentation of soil grains in water. It is expressed by Stokes Law, which says the velocity of the soil sedimentation is based on the soil particles shape, size, weight, and viscosity of the water. Thus, the hydrometer analysis measures the change in specific gravity of a soil-water suspension as soil particles settle out over time. See ASTM D422 - *Standard Test Method for Particle-Size Analysis of Soils* (AASHTO T88 - *Standard Method of Test for Particle Size Analysis of Soils*).

5.4.2 Moisture Content

The moisture content (*w*) is defined as the ratio of the weight of water in a sample to the weight of solids. The weight of the solids must be oven dried and is considered as weight of dry soil. Organic soils can have the water content determined, but must be dried at a lower temperature for the weight of dry soil to prevent degradation of the organic matter. See ASTM D2216 - *Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass* (AASHTO T265 - *Standard Method of Test for Laboratory Determination of Moisture Content of Soils*).

5.4.3 Atterberg Limits

The Atterberg Limits are different descriptions of the moisture content of fine-grained soils as it transitions between a solid to a liquid-state. For classification purposes the two primary Atterberg Limits used are the plastic limit (PL) and the liquid limit (LL). The plastic index (PI) is also calculated for soil classification.

5.4.3.1 Plastic Limit

The plastic limit (PL) is the moisture content at which a soil transitions from being in a semisolid state to a plastic state. Tests shall be performed in accordance with ASTM D4318 - *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils* (AASHTO T90 - *Standard Method of Test for Determining the Plastic Limit and Plasticity Index of Soils*).

5.4.3.2 Liquid Limit

The liquid limit (LL) is defined as the moisture content at which a soil transitions from a plastic state to a liquid state. Tests shall be performed in accordance with ASTM D4318 - *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils* (AASHTO T89 - *Standard Method of Test for Determining the Liquid Limit of Soils*).

5.4.3.3 Plasticity Index

The plasticity index (PI) is defined as the difference between the liquid limit and the plastic limit of a soil. The PI represents the range of moisture contents within which the soil behaves as a plastic solid.

5.4.4 Specific Gravity of Soils

The specific gravity of soil, G_s, is defined as the ratio of the unit weight of a given material to the unit weight of water. The procedure is applicable only for soils composed of particles smaller than the No. 4 sieve (4.75 mm). See ASTM D854 - *Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer* (AASHTO T100 - *Standard Method of Test for Specific Gravity of Soils*). If the soil contains particles larger than the No. 4 sieve (4.75 mm), use ASTM C127- *Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate*.

5.4.5 Strength Tests

The shear strength is the internal resistance per unit area that the soil can handle before failure and is expressed as a stress. There are two components of shear strength, cohesive element (expressed as the cohesion, c, in units of force/unit area) and frictional element (expressed as the angle of internal friction, ϕ). These parameters are expressed in the form of total stress (c, ϕ) or effective stress (c', ϕ'). The total stress on any subsurface element is produced by the overburden pressure plus any applied loads. The effective stress equals the total stress minus the pore water pressure. The common methods of ascertaining these parameters in the laboratory are discussed below. All of these tests are normally performed on undisturbed samples, but may also be performed on remolded samples.

5.4.5.1 Unconfined Compression Tests

The unconfined compression test is a quick method of determining the value of undrained cohesion (c_u) for clay soils. The test involves a clay specimen with no confining pressure and an axial load being applied to observe the axial strains corresponding to various stress levels. The stress at failure is referred to as the unconfined compression strength. The c_u is taken as one-half the unconfined compressive strength, q_u. See ASTM D2166 - *Standard Test Method for Unconfined Compressive Strength of Cohesive Soil* (AASHTO T208 - *Standard Method of Test for Unconfined Compressive Strength of Cohesive Soil*).

5.4.5.2 Triaxial Compression Tests

The triaxial compression test is a more sophisticated testing procedure for determining the shear strength of a soil. The test involves a soil specimen subjected to an axial load until failure while also being subjected to confining pressure that approximates the in-situ stress conditions. There are three types of triaxial tests which are described below.

5.4.5.2.1 Unconsolidated-Undrained (UU), or Q Test

In unconsolidated-undrained tests, the specimen is not permitted to change its initial water content before or during shear. The results are total stress parameters. This test is used primarily in the calculation of immediate embankment stability during quick-loading conditions. Refer to ASTM D2850 - *Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils* (AASHTO T296 - *Standard Method of Test for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression*).

5.4.5.2.2 Consolidated-Undrained (CU), or R Test

The consolidated-undrained test is the most common type of triaxial test. This test allows the soil specimen to be consolidated under a confining pressure prior to shear. After the pore water pressure is dissipated, the drainage line will be closed and the specimen will be subjected to shear. Several tests on similar specimens with varying confining pressures may have to be made to determine the shear strength parameters. See ASTM D4767 - *Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils* (AASHTO T297 - *Standard Method of Test for Consolidated, Undrained Triaxial Compression Test on Cohesive Soils*).

5.4.5.2.3 Consolidated-Drained (CD), or S Test

The consolidated-drained test is similar to the consolidated-undrained test except that drainage is permitted during shear and the rate of shear is very slow. Thus, the buildup of excess pore pressure is prevented. Again, several tests on similar specimens will be conducted to determine the shear strength parameters. This test is used to determine parameters for calculating long-term stability of embankments. Refer to ASTM WK3821 - *New Test Method for Consolidated Drained Triaxial Compression Test for Soils*.

5.4.5.3 Direct Shear

The direct shear test is the oldest and simplest form of shear test. A soil sample is placed in a metal shear box and undergoes a horizontal force. The soil fails by shearing along a plane when the force is applied. The test can be performed either in stress-controlled or strain-controlled. In addition the test is typically performed as consolidated-drained test on cohesionless soils. See ASTM D3080 - *Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions* (AASHTO T236 - *Standard Method of Test for Direct Shear Test of Soils Under Consolidated Drained Conditions*).

5.4.5.4 Miniature Vane Shear (Torvane) and Pocket Penetrometer

The miniature vane shear and the pocket penetrometer tests are performed to obtain undrained shear for plastic cohesive soils. Both of these tests consist of hand-held devices that are pushed into the sample and either a torque resistance (Torvane) or a tip resistance (pocket penetrometer) is measured. They can be performed in the lab or in the field. See ASTM D4648 - *Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil* for the miniature vane shear test only.

5.4.6 Consolidation Test

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 elastic compression, primary and secondary consolidation. Sandy (coarse-grained) soils tend to be relatively permeable and will therefore, undergo settlement much faster. The amount of elastic compression settlement can vary depending on the soil type; however, the time for this settlement to occur is relatively quick and will normally occur during construction. Clayey (fine-grained) soils have a much lower permeability and will, therefore, take longer to settle. Clayey soils undergo elastic compression during the initial stages of loading (i.e. the soil particles rearrange due to the loading). After elastic compression, clayey soils enter primary consolidation. Saturated clayey soils have a lower coefficient of permeability, thus the excess pore water pressure generated by loading will gradually dissipate over a longer period of time. Therefore in saturated clays, the amount and rate of settlement is of great importance in construction. For example, an embankment may settle until a gap exists between an approach and a bridge abutment. The calculation of settlement involves many factors, including the magnitude of the load, the effect of the load at the depth at which compressible soils exist, the water table, and characteristics of the soil itself. Consolidation testing is performed to ascertain the nature of these characteristics.

The most often used method of consolidation testing is the one-dimensional test. The consolidation test unit consists of a consolidometer (oedometer) and a loading device. The soil sample is placed between two porous stones, which permit drainage. Load is applied incrementally and is typically held up to 24 hours. The test measures the height of the specimen after each loading is applied. The results are plotted on a time versus deformation log scale plot. From this curve, two parameters can be derived: coefficient of consolidation (C_v) and coefficient of secondary compression (C_α). These parameters are used to predict the rate of primary settlement and the amount of secondary consolidation.

After the time-deformation plots are obtained, the void ratio and the strain can be calculated. Two more plots can be presented; an e-log p curve, which plots void ratio (e) as a function of the log of pressure (p), or an ε-log p curve where ε equals percent strain. The parameters necessary for settlement calculation can be derived from the e-log p curve and are: compression index (C_c), recompression index (C_r), preconsolidation pressure (P_c), and initial void ratio (e_o). Alternatively, the ε-log p curve provides the compression index ($C_{\varepsilon c}$), the recompression index ($C_{\varepsilon r}$), and the preconsolidation pressure (P_c).

To evaluate the recompression parameters of the sample, an unload/reload cycle can be performed during the loading schedule. To better evaluate the recompression parameters for overconsolidated clays, the unload/reload cycle may be performed after the preconsolidation pressure has been defined. After the maximum loading has been reached, the loading is removed in appropriate decrements. See ASTM D2435 - *Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading* (AASHTO T216 - *Standard Method of Test for One-Dimensional Consolidation Properties of Soils*).

For soils that are high in organic material and highly compressible inorganic soils, secondary consolidation is more important than primary consolidation.

For high organic materials (organic content greater than 50%), research sponsored by the Florida Department of Transportation has shown that the end of primary consolidation occurs quickly in the laboratory and field, and that a major portion of the total settlement is due to secondary consolidation (creep). As a result, differentiating between primary consolidation and creep settlement can be very difficult and generate misleading results. To analyze results from one-dimensional consolidation tests for these types of materials, use the Square Root (Taylor) Method to identify the end of primary consolidation for each load sequence. In addition, each load sequence must be maintained for at least 24 hours to identify a slope for the secondary consolidation portion of the settlement versus time plot.

5.4.7 Organic Content

Organic soils demonstrate very poor engineering characteristics, most notably low strength and high compressibility. In the field these soils can usually be identified by their dark color, musty odor and low unit weight. The most used laboratory test for design purposes is the Ignition Loss test, which measures how much of a sample's mass burns off when placed in a muffle furnace. The results are presented as a percentage of the total sample mass. Tests shall be performed in accordance with ASTM D2974 - *Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils* (AASHTO T267 - *Standard Method of Test for Determination of Organic Content in Soils by Loss on Ignition*).

5.4.8 Shrinkage and Swell

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. Shrinkage can cause soil to pull away from structure thus reducing the bearing area or causing settlement of the structure beyond that predicted by settlement analysis. Swelling of the soil can cause an extra load to be applied to the structure that was not accounted for in design. Therefore, the potential for shrinkage and swelling should be determined for soils that have high plasticity.

5.4.8.1 Shrinkage

These tests are performed to determine the limits of a soil's tendency to lose volume during decreases in moisture content. The shrinkage limit (SL) is presented as a percentage in moisture content, at which the volume of the soil mass ceases to change. See ASTM D427 - *Test Method for Shrinkage Factors of Soils by the Mercury Method* (AASHTO T92 - *Standard Method of Test for Determining the Shrinkage Factors of Soils*).

5.4.8.2 Swell

There are certain types of soils that can swell, particularly clay in the montmorillonite family. Swelling occurs when the moisture is allowed to increase causing the clay soil to increase in volume. There are a number of reasons for this to occur: the elastic rebound of the soil grains, the attraction of the clay mineral for water, the electrical repulsion of the clay particles and their adsorbed cations from one another, or the expansion of the air trapped in the soil voids. In the montmorillonite family, adsorption and repulsion predominate and this can cause swelling. Testing for swelling is difficult, but can be done. It is recommended that these soils not be used for roadway construction. The swell potential can be estimated from the test methods shown in ASTM D4546 - *Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils* (AASHTO T258 - *Standard Method of Test for Determining Expansive Soils*).

5.4.9 Permeability

Permeability, also known as hydraulic conductivity, has the same units as velocity and is generally expressed in ft/min or m/sec. Coefficient of permeability is dependent on void ratio, grain-size distribution, pore-size distribution, roughness of mineral particles, fluid viscosity, and degree of saturation. There are three standard laboratory test procedures for determining the coefficient of permeability soil, constant and falling head tests and flexible wall tests.

5.4.9.1 Constant Head Test

In the constant head test, water is poured into a sample of soil, and the difference of head between the inlet and outlet remains constant during the testing. After the flow of water becomes constant, water that is collected in a flask is measured in quantity over a time period. This test is more suitable for coarse-grained soils that have a higher coefficient of permeability. See ASTM D2434 - *Standard Test Method for Permeability of Granular Soils (Constant Head)* (AASHTO T215 - *Standard Method of Test for Permeability of Granular Soils (Constant Head)*).

5.4.9.2 Falling Head Test

The falling head test uses a similar procedure to the constant head test, but the head is not kept constant. The permeability is measured by the decrease in head over a specified time. This test is more appropriate for fine-grained soils. Tests shall be performed in accordance with ASTM D5856 - *Standard Test Method for Measurement of Hydraulic Conductivity of Porous Material Using a Rigid-Wall, Compaction-Mold Permeameter*.

5.4.9.3 Flexible Wall Permeability

For fine-grained soils, tests performed using a triaxial cell are generally preferred. In-situ conditions can be modeled by application of an appropriate confining pressure. The sample can be saturated using back pressuring techniques. Water is then allowed to flow through the sample and measurements are taken until steady-state conditions occur. Tests shall be performed in accordance with ASTM D 5084 - *Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter*.

5.4.10 Compaction Tests

There are two types of tests that can determine the optimum moisture content and maximum dry density of a soil; the Standard Proctor and the Modified Proctor. The results of the tests are used to determine appropriate methods of field compaction and to provide a standard by which to judge the acceptability of field compaction.

The results of the compaction tests are typically plotted as dry density versus moisture content. Tests have shown that moisture content has a great influence on the degree of compaction achieved by a given type of soil. In addition to moisture content, there are other important factors that affect compaction. The soil type has a great influence because of its various classifications, such as grain size distribution, shape of the soil grains, specific gravity of soil solids, and amount and type of clay mineral present. The compaction energy also has an affect because it too has various conditions, such as number of blows, number of layers, weight of hammer, and height of the drop.

5.4.10.1 Standard Proctor

This test method uses a 5-1/2-pound rammer dropped from a height of 12 inches. The sample is compacted in three layers. See ASTM D698 - *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3))* (AASHTO T99 - *Standard Method of Test for Moisture-Density Relations of Soils Using a 2.5-kg (5.5-lb) Rammer and a 305-mm (12-in.) Drop*).

5.4.10.2 Modified Proctor

This test method uses a 10-pound rammer dropped from a height of 18 inches. The sample is compacted in five layers. See ASTM 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))* (AASHTO T180 - *Standard Method of Test for Moisture-Density Relations of Soils Using a 4.54-kg (10-lb) Rammer and a 457-mm (18-in.) Drop*).

5.4.11 Relative Density Tests

The relative density tests are most commonly used for granular or unstructured soils. It is used to indicate the in-situ denseness or looseness of the granular soil. In comparison, Proctor tests often do not produce a well-defined moisture-density curve for cohesionless, free-draining soils. Therefore relative density is expressed in terms of maximum and minimum possible dry unit weights and can be used to measure compaction in the field.

5.4.11.1 Maximum Index Density

In this test, soil is placed in a mold of known volume with a 2-psi surcharge load applied to it. The mold is then vertically vibrated at a specified frequency for a specified time. At the end of the vibrating period, the maximum index density can be calculated using the weight of the sand and the volume of the sand. See ASTM D4253 - *Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table*.

5.4.11.2 Minimum Index Density

The test procedure requires sand being loosely poured into a mold at a designated height. The minimum index density can be calculated using the weight of the sand required to fill the mold and the volume of the mold. See ASTM D4254 - *Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density*.

5.4.12 Electro-Chemical Tests

Electro-chemical tests provide quantitative information related to the aggressiveness of the subsurface environment, the surface water environment, and the potential for deterioration of foundation materials. Electro-chemical testing includes pH, resistivity, sulfate, and chloride contents. The electro-chemical tests should be performed on soil samples. In addition, surface water should also be tested in coastal regions where the potential intrusion of brackish (salt water) water may occur in tidal streams.

5.4.12.1 pH Testing

pH testing is used to determine the acidity or alkalinity of the subsurface or surface water environments. Acidic or alkaline environments have the potential for being aggressive on structures placed within these environments. Soil samples collected during the normal course of a subsurface exploration should be used for pH testing. Surface water samples shall be obtained in general accordance with standards published by the South Carolina Department of Health and Environmental Control. The pH of soils shall be determined using ASTM D4972 – *Standard Test Method for pH of Soils* (uses an aqueous method); ASTM G51 – *Standard Test Method for Measuring pH of Soils for Use in Corrosion Testing* (uses a nonaqueous method); or AASHTO T289 - *Standard Method of Test for Determining pH of Soil for Use in Corrosion Testing*. Any of these methods may be used; however, the laboratory shall be certified to perform the appropriate test method and shall indicate the method used on the laboratory results report. The surface water samples shall have the pH determined using ASTM D1293 – *Standard Test Methods for pH of Water*.

5.4.12.2 Resistivity Testing

Resistivity testing is used to determine the electric conduction potential of the subsurface environment. The ability of soil to conduct electricity can have a significant impact on the corrosion of steel piling. If a soil has a high potential for conducting electricity, then sacrificial anodes may be required on the structure. This type of testing can be performed in the laboratory or in the field. For the field testing procedure see Section 5.3.7.6 of this Manual. Resistivity shall be determined using ASTM G57 – S*tandard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method* or AASHTO T288 – *Standard Method of Test for Determining Minimum Laboratory Soil Resistivity*. The resistivity of surface water samples can be determined using ASTM D1125 – *Standard Test Method for Electrical Conductivity and Resistivity of Water*. As in pH testing, the surface water sample shall be obtained in accordance with sampling procedures prepared by the South Carolina Department of Health and Environmental Control.

5.4.12.3 Chloride Testing

Subsurface soils and surface water should be tested for chloride if the presence of sea or brackish water is suspected. Chloride testing for soils shall be determined using AASHTO T291 – *Standard Method of Test for Determining Water-Soluble Chloride Ion Content in Soil*. The chloride testing for the surface water shall be performed in accordance with ASTM D512 – *Standard Test Methods for Chloride Ion in Water*.

5.4.12.4 Sulfate Testing

Subsurface soils and surface water should be tested for sulfate. Sulfate testing for soils shall be determined using AASHTO T290 – *Standard Method of Test for Determining Water-Soluble Sulfate Ion Content in Soil*. The sulfate testing for the surface water shall be performed in accordance with ASTM D516 – *Standard Test Methods for Sulfate Ion in Water*.

5.4.13 Rock Cores

Rock coring is conducted when a soil boring encounters material that has a standard penetration resistance, N, exceeding 100 blows and is termed auger refusal. Typically rock coring is conducted to 10 feet into rock. At each core run, the length of the rock sample obtained and the distance the core run is drilled will give a recovery ratio. The recovery ratio is expressed in percentage with 100% being intact rock and 50% or below as highly fractured rock. Another way to evaluate rock is rock quality designation (RQD) which is also expressed in percentage. The RQD allows the Engineer to determine if compressive strengths can be performed at each core run. It is highly recommended to have rock coring done as close to the proposed shaft or pile as possible. South Carolina geology can have a rock formation that changes in a number of feet along the length or the width of the bridge.

5.4.13.1 Unconfined Compression Test

This test is performed on intact rock core specimens, usually with a rock sample length of 2 times the diameter. The specimen undergoes a confined compression or uniaxial compression. After the test, it provides data determining the strength of the rock, namely the uniaxial strength, shear strengths at varying pressures and varying temperatures, angle of internal friction, (angle of shearing resistance), and cohesion intercept. See ASTM D7012 - *Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures*.

5.5 QUALITY ASSURANCE/QUALITY CONTROL

The Quality Assurance/Quality Control (QA/QC) of the field and laboratory testing procedures/methods can have a significant impact on the results obtained from the testing. Therefore, all field and laboratory testing will require a QA/QC plan to be developed, maintained and implemented. The QA/QC plan shall follow the appropriate national, state or approved industrial standards.

5.5.1 Field Testing QA/QC Plan

All field testing shall be performed in accordance with an approved QA/QC plan. The plan shall at a minimum establish the calibration schedule for the equipment, the method of calibration and provide circumstances when calibration is required differently from the regularly scheduled calibration. The QA/QC plan shall be approved by the PCS/GDS with concurrence by the Office of Materials and Research.

5.5.2 Laboratory Testing QA/QC Plan

All laboratories conducting geotechnical testing shall be AASHTO Materials Reference Laboratory (AMRL) certified. The laboratories shall only conduct those tests for which the laboratory is certified. If the laboratory is not certified to conduct the test, the laboratory may contract to another laboratory that is certified. If no laboratory is certified, then a QA/QC plan for that particular test shall be developed and submitted to the Department for review and approval prior to testing. The QA/QC plan shall indicate which test method is being followed, the most recent calibration of the laboratory equipment to be used and the qualifications of the personnel performing the test. For tests where there is not an established ASTM, AASHTO or State testing standard, then the laboratory may use a testing method established by another Federal or State agency. The use of other agency standards shall be approved in writing by the Department prior to conducting the test. The laboratory requesting the use of another agency standard shall prove proficiency in the standard as well as submitting a QA/QC plan for the test method.

5.6 REFERENCES

Application of Geophysical Methods to Highway Related Problems, FHWA-IF-94-021, August 2004.

ASTM International 2006, 'D7012-04 Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures'.

Das, M. Braja, Principles of Geotechnical Engineering, 3rd edition, PWS Publishing Company, Boston MA, 1994.

Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5, FHWA-IF-02-034, April 2002.

Sowers, George F., Introductory Soil Mechanics and Foundations: Geotechnical Engineering, 4th edition, Macmillan Publishing Co., Inc., New York, NY, 1970.

Sowers, George F. and Hedges, Charles S, Dynamic Cone for Shallow In-Situ Penetration Testing, Vane Shear and Cone Penetration Resistance Testing of In-Situ Soils, ASTM STP399, 1966

Spangler, Merlin G., and Handy, Richard L., Soil Engineering, 4th edition, Harper & Row, Publishers, New York, NY, 1982.

5.7 SPECIFICATIONS AND STANDARDS

Table 5-1, Specifications and Standards

Subject	ASTM	AASHTO	SCDOT
Limerock Bearing Ratio	-	-	-
Resilient Modulus of Soils and Aggregate Materials	-	T307	-
Absorption and Bulk Specific Gravity of Dimension Stone	C97	-	-
Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate	C127	T85	-
Standard Test Method for Particle-Size Analysis of Soils	D422	T88	-
Test Method for Shrinkage Factors of Soils by the Mercury Method	D427	T92	-
Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3))	D698	T99	-
Standard Test Method for Specific Gravity of Soils	D854	T100	-
Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3))	D1557	T180	SC-T-140
Standard Test Method for Unconfined Compressive Strength of Cohesive Soil	D2166	T208	-
Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock	D2216	T265	-
Standard Test Method for Permeability of Granular Soils (Constant Head)	D2434	T215	-
Standard Test Method for One-Dimensional Consolidation Properties of Soils	D2435	T216	-
Standard Test Method for Triaxial Compressive Strength of Undrained Rock Core Specimens Without Pore Pressure Measurements	D2664	-	-
Standard Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression	D2850	T296	-
Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens	D2938	-	SC-T-39

Table 5-1 (Continued), Specifications and Standards (Continued)

Subject	ASTM	AASHTO	SCDOT
Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils	D2974	T267	-
Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	D3080	T236	-
Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens	D3967	-	-
Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading	D4186	-	-
Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table	D4253	-	-
Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density	D4254	-	-
Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	D4318	T89 & T90	-
Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils	D4546	T258	-
Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil	D4648	-	-
Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	D4767	T297	-
Standard Practices for Preserving and Transporting Rock Core Samples	D5079	-	-
Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter	D5084	-	-
Standard Test Method for pH of Soils	D4972	T289	-
Standard Test Method for pH of Soils for use in Corrosion Testing	G51	T289	-
Standard Test Methods for pH of Water	D1293	-	-
Standard Test Method for Determining Soil Resistivity	G57	T288	-
Standard Test Method for Electrical Conductivity and Resistivity of Water	D1125	-	-
Standard Test Method for Determining Chloride	D512	T291	-
Standard Test Method for Determining Sulfate	D516	T290	

Chapter 6

MATERIAL DESCRIPTION, CLASSIFICATION, AND LOGGING

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Table of Contents

List of Tables

List of Figures

CHAPTER 6

MATERIAL DESCRIPTION, CLASSIFICATION, AND LOGGING

6.1 INTRODUCTION

Geomaterials (soil and rock) are naturally occurring materials used in highway construction by SCDOT. Understanding soil and rock behavior is critical to the completion of any project designed or constructed by SCDOT. Soil and rock classification is an essential element of understanding the behavior of geomaterials. During field exploration, a log must be kept of the materials encountered. A field engineer, a geologist, or the driller usually keeps the field log. Details of the subsurface conditions encountered, including basic material descriptions and details of the drilling and sampling methods should be recorded. See ASTM D5434 - *Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock*. Upon delivery of the samples to the laboratory, an experienced technician, engineer or geologist will generally verify or modify material descriptions and classifications based on the results of laboratory testing and/or detailed visual-manual inspection of samples.

Material descriptions, classifications, and other information obtained during the subsurface explorations are heavily relied upon throughout the remainder of the investigation program and during the design and construction phases of a project. It is therefore necessary that the method of reporting this data be standardized. Records of subsurface explorations should follow as closely as possible the standardized format presented in this chapter.

This chapter is divided into two primary sections, the first is associated with the description and classification of soil and the second section will discuss the description and classification of rock. The soil description and classification section will discuss the two soil classification systems used by SCDOT (i.e. the USCS and AASHTO).

6.2 SOIL DESCRIPTION AND CLASSIFICIATION

A detailed description for each material stratum encountered should be included on the log. The extent of detail will be somewhat dependent upon the material itself and on the purpose of the project. However, the descriptions should be sufficiently detailed to provide the engineer with an understanding of the material present at the site. Since it is rarely possible to test all of the samples obtained during an exploration program, the descriptions should be sufficiently detailed to permit grouping of similar materials and aide in the selection of representative samples for testing.

Soils should be described with regard to soil type, color, relative density/consistency, etc. The description should match the requirements of the Unified Soil Classification System (USCS) and AASHTO. A detailed soil description should include the following items, in order:

1. Relative Density/Consistency
2. Moisture Condition
3. Color
4. Particle Angularity and Shape (coarse-grained)
5. Hydrochloric (HCl) Reaction

6. Cementation
7. Gradation (coarse-grained)
8. Plasticity (fine-grained)
9. Classification (USCS and AASHTO)
10. Other pertinent information

6.2.1 Relative Density/Consistency

Relative density refers to the degree of compactness of a coarse-grained soil. Consistency refers to the stiffness of a fine-grained soil. When evaluating subsurface soil conditions using correlations based on safety hammer SPT tests, SPT N-values obtained using an automatic hammer shall be corrected for energy to produce the equivalent safety hammer SPT N-value (see Chapter 7 for correction). However, only actual field recorded (uncorrected) SPT N-values shall be included on the Soil Test Boring Log.

Standard Penetration Test N-values (blows per foot) are usually used to define the relative density and consistency as follows:

Table 6-1, Relative Density / Consistency Terms

Relative Density[1,2]			Consistency[1,3]		
Descriptive Term	**Relative Density**	**SPT Blow Count (bpf)[4]**	**Descriptive Term**	**Unconfined Compression Strength (q_u) (tsf)**	**SPT Blow Count (bpf)[4]**
Very Loose	0 to 15%	< 4	Very Soft	<0.25	<2
Loose	16 to 35%	5 to 10	Soft	0.26 to 0.50	3 to 4
Medium Dense	36 to 65%	11 to 30	Firm	0.51 to 1.00	5 to 8
Dense	66 to 85%	31 to 50	Stiff	1.01 to 2.00	9 to 15
Very Dense	86 to 100%	>51	Very Stiff	2.01 to 4.00	16 to 30
			Hard	>4.01	> 31
[1]For Classification only, not for design					
[2]Applies to coarse-grained soils (major portion retained on No. 200 sieve)					
[3]Appiles to fine-grained soils (major portion passing No. 200 sieve)					
[4]bpf – blows per foot of penetration					

6.2.2 Moisture Condition

The in-situ moisture condition shall be determined using the visual-manual procedure. The moisture condition is defined using the following terms:

Table 6-2, Moisture Condition Terms

Descriptive Term	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually in coarse-grained soils below the water table

6.2.3 Soil Color

The color of the soil shall be determined using the Munsell color chart and shall be described while the soil is still at or near the in-situ moisture condition. The color designation shall be provided at the end of the soils description.

6.2.4 Particle Angularity and Shape

Coarse-grained soils are described as angular, subangular, subrounded, or rounded. Gravel, cobbles, and boulders can be described as flat, elongated, or flat and elongated. Descriptions of fine-grained soils will not include a particle angularity or shape.

Table 6-3, Particle Angularity and Shape

Descriptive Term	Criteria
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges
Flat	Particles with a width to thickness ratio greater than 3
Elongated	Particles with a length to width ratio greater than 3
Flat and Elongated	Particles meeting the criteria for both Flat and Elongated

6.2.5 HCl Reaction

The terms presented below describe the reaction of soil with HCl. Since calcium carbonate is a common cementing agent, a report of its presence on the basis of the reaction with dilute hydrochloric acid is important.

Table 6-4, HCl Reaction

Descriptive Term	Criteria
None	No visible reaction
Weakly	Some reaction, with bubbles forming slowly
Strongly	Violent reaction, with bubbles forming immediately

6.2.6 Cementation

The terms presented below describe the cementation of intact coarse-grained soils.

Table 6-5, Cementation

Descriptive Term	Criteria
Weakly Cemented	Crumbles or breaks with handling or little finger pressure
Moderately Cemented	Crumbles or breaks with considerable finger pressure
Strongly Cemented	Will not crumble or break with finger pressure

6.2.7 Gradation

The classification of soil is divided into two general categories based on gradation, coarse-grained and fine-grained soils. Coarse-grained soils (gravels and sands) have more than 50 percent (by weight) of the material retained on the No. 200 sieve, while fine-grained soils (silts and clays) have more than 50 percent of the material passing the No. 200 sieve. Gravels and sands are typically described in relation to the particle size of the grains. Silts and clays are typically described in relation to plasticity. The primary constituents are identified considering grain size distribution. In addition to the primary constituent, other constituents which may affect the engineering properties of the soil should be identified. Secondary constituents are generally indicated as modifiers to the principal constituent (i.e., sandy clay or silty gravel, etc.). Other constituents can be included in the description using the terminology of ASTM D2488 through the use of terms such as trace (<5%), few (5-10%), little (15-25%), some (30-45%), and mostly (50-100%).

6.2.7.1 Coarse-Grained Soils

Coarse-grained soils are those soils with more than 50 percent by weight retained on or above the No. 200 sieve. Well- and poorly-graded only apply to coarse-grained soils. The difference between well- and poorly-graded depends upon the Coefficient of Curvature (C_c) and the Coefficient of Uniformity (C_u).

$$\mathbf{C_c = \frac{(D_{30})^2}{[(D_{10})(D_{60})]}}$$ **Equation 6-1**

$$\mathbf{C_u = \frac{(D_{60})}{(D_{10})}}$$ **Equation 6-2**

Where,

D_{10} = diameter of particle at 10% finer material
D_{30} = diameter of particle at 30% finer material
D_{60} = diameter of particle at 60% finer material

The particle size for gravels and sands are provided in Table 6-6 and the adjectives used for describing the possible combinations of particle size are provided in Table 6-7.

Table 6-6, Coarse-Grained Soil Constituents

Soil Component	Grain Size
Gravel	
Coarse	3” to ¾”
Fine	¾” to No. 4 sieve
Sand	
Coarse	No. 4 to No. 10 sieve
Medium	No. 10 to No. 40 sieve
Fine	No. 40 to No. 200 sieve

Table 6-7, Adjectives For Describing Size Distribution

Particle-Size Adjective	Abbreviation	Size Requirements
Coarse	c	< 30% m/f Sand or < 12% f Gravel
Coarse to medium	c/m	< 12% f Sand
Medium to fine	m/f	< 12% c Sand and > 30% m Sand
Fine	f	< 30% m Sand or < 12% c Gravel
Coarse to fine	c/f	> 12% of each size

6.2.7.2 Fine-Grained Soils

Fine-grained soils are those soils with more than 50 percent passing the No. 200 sieve. These materials are defined using moisture-plasticity relationships developed in the early 1900's by the Swedish soil scientist A. Atterberg. Atterberg developed five moisture-plasticity relationships, of which 2 are used in engineering practice and are known as Atterberg Limits. These limits are the liquid limit (LL) and the plastic limit (PL). The plastic limit is defined as the moisture content at which a 1/8" diameter thread can be rolled out and at which the thread just begins to crumble. The liquid limit is the moisture content at which a soil will flow when dropped a specified distance and a specified number of times. In addition, the plastic index (PI) is the range between the plastic limit and the liquid limit (LL-PL). The Plasticity Chart, Figure 6-1, is used to determine low and high plasticity and whether a soil will be Silt or Clay. Table 6-8 provides the adjectives used to describe plasticity and the applicable plasticity range.

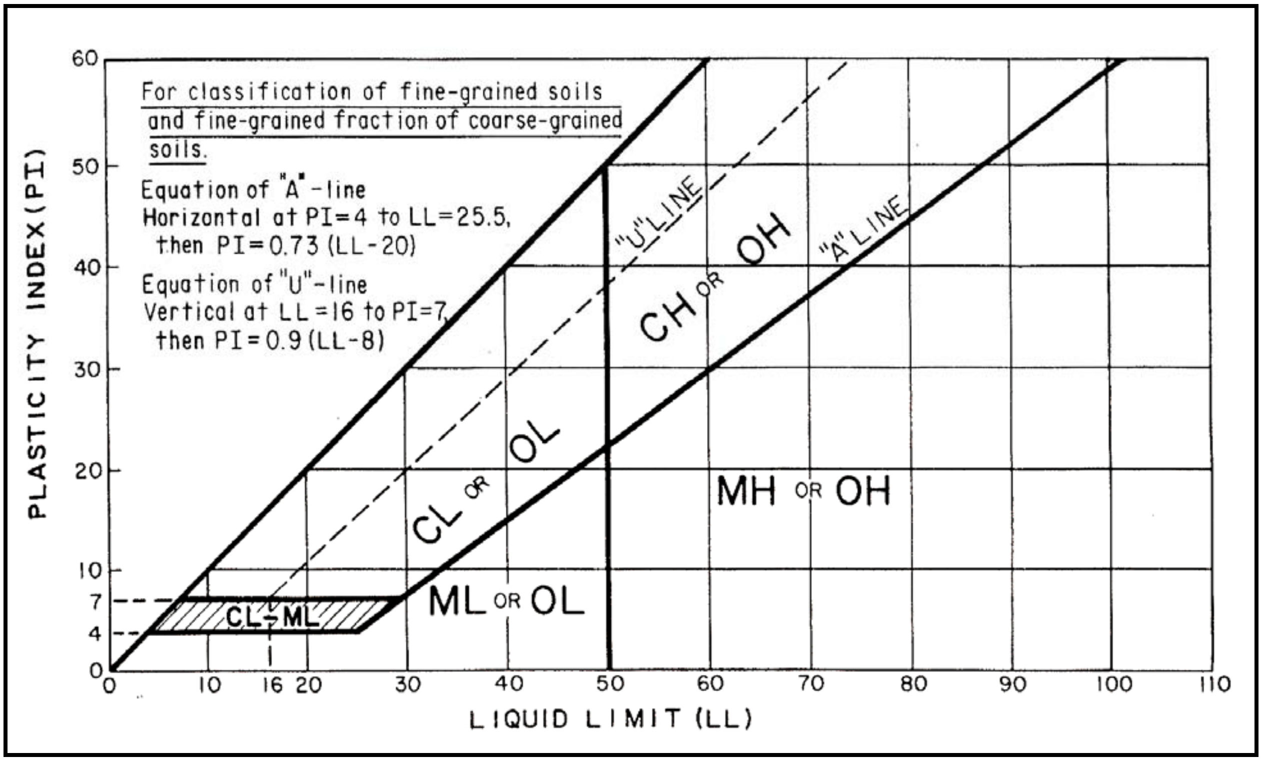


Figure 6-1, Plasticity Chart

Table 6-8, Soil Plasticity Descriptions

PI Range	Adjective	Dry Strength
0	non-plastic	none – crumbles into powder with mere pressure
1 – 10	low plasticity	low – crumbles into powder with some finger pressure
11 – 20	medium plasticity	medium – breaks into pieces or crumbles with considerable finger pressure
21 – 40	high plasticity	high – cannot be broken with finger pressure
> 41	very plastic	very high – cannot be broken between thumb and a hard surface

6.2.8 Unified Soil Classification System (USCS)

Dr. A. Casagrande developed the USCS for the classification of soils used to support Army Air Corps bomber bases. This system incorporates textural (grain-size) characteristics into the engineering classification. The system has 15 different potential soil classifications with each classification having a two-letter designation. The basic letter designations are listed Table 6-9.

Table 6-9, Letter Designations

Letter Designation	Meaning	Letter Designation	Meaning
G	Gravel	O	Organic
S	Sand	W	Well-graded
M	Non-plastic or low plasticity fines (Silt)	P	Poorly-graded
C	Plastic fines (Clay)	L	Low liquid limit
Pt	Peat	H	High liquid limit

The classification of soil is divided into two general categories, coarse-grained and fine-grained soils. Coarse-grained soils (gravels and sands) have more than 50 percent (by weight) of the material retained on the No. 200 sieve, while fine-grained soils (silts and clays) have more than 50 percent of the material passing the No. 200 sieve. Gravels and sands are typically described in relation to the particle size of the grains (See Section 6.2.1.7.1 – Coarse-Grained Soils). Silts and clays are typically described in relation to plasticity (see Section 6.2.1.7.2 – Fine-Grained Soils).

In many soils, two or more soil types are present. When the percentage of the minor soil type is equal to or greater than 30 percent and less than 50 percent of the total sample (by weight), the minor soil type is indicated by adding a “y” to its name; i.e. Sandy SILT, Silty SAND, Silty CLAY, etc.

Figures 6-2, 6-3, 6-4, 6-5, and 6-6 provide the flow charts for the classification of coarse- and fine-grained soils using the USCS.

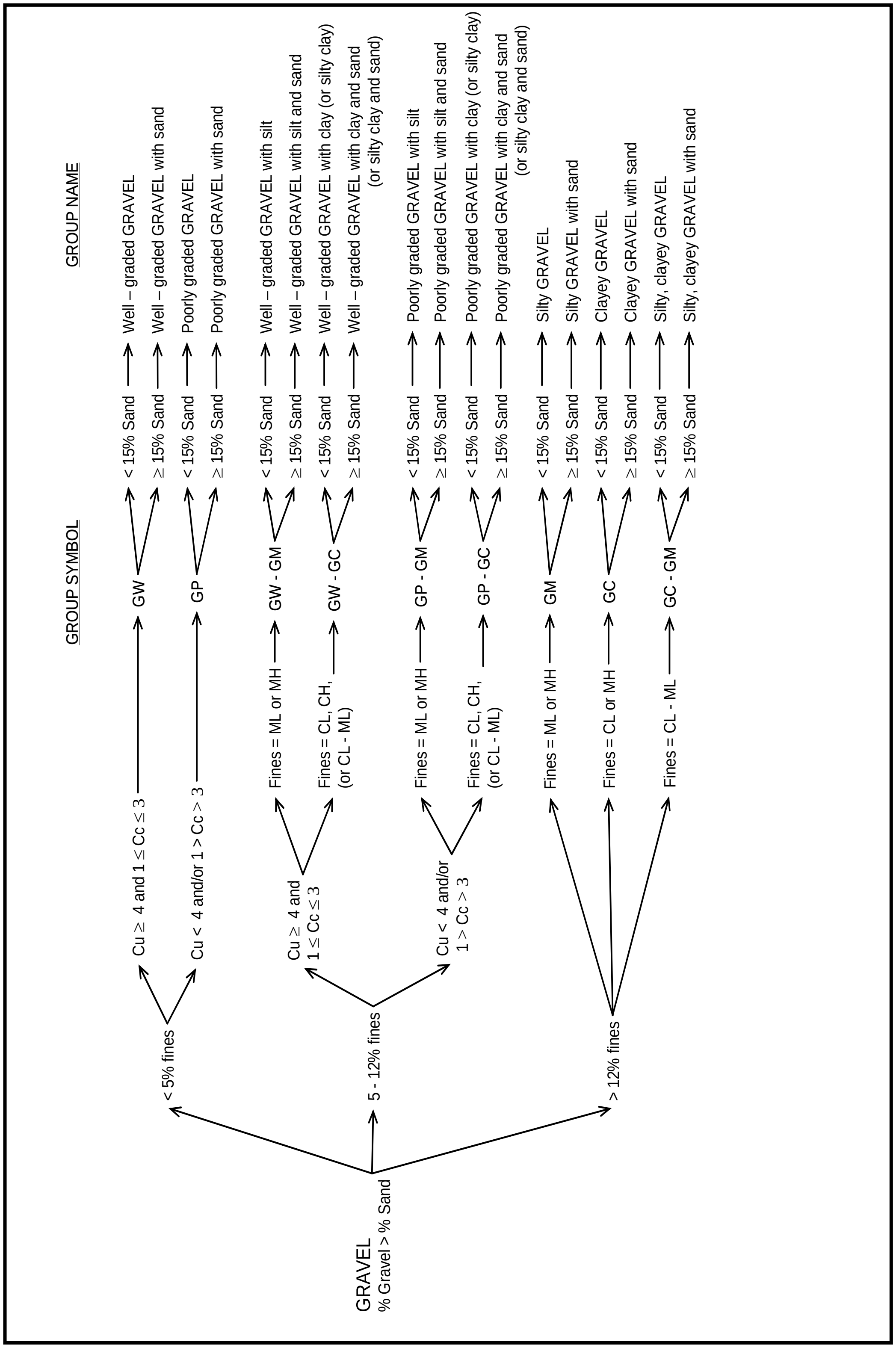


Figure 6-2, Group Symbol and Group Name Coarse-Grained Soils (Gravel)
(Subsurface Investigations – Geotechnical Site Characterization – May 2002)

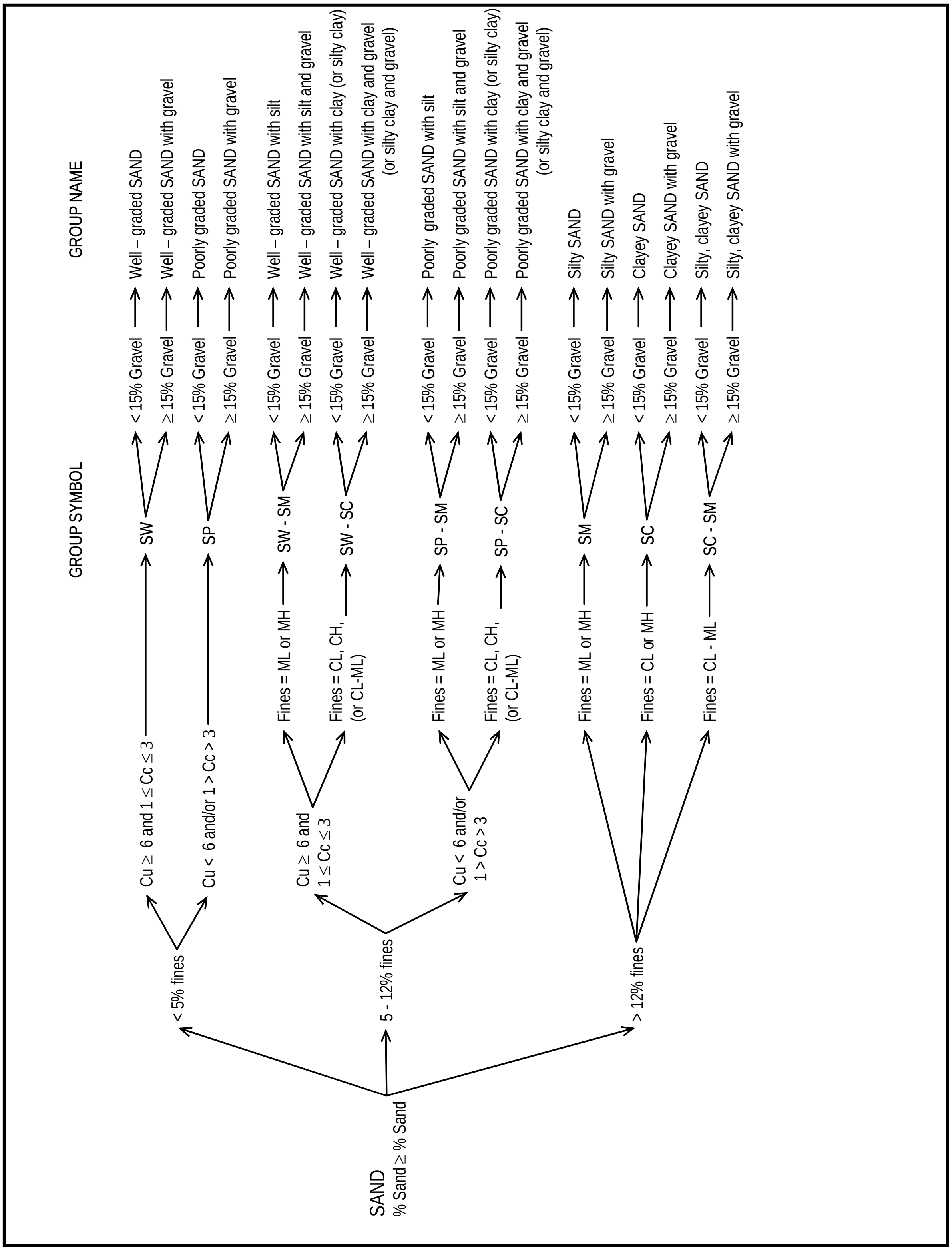


Figure 6-3, Group Symbol and Group Name for Coarse-Grained Soils (Sand) (Subsurface Investigations – Geotechnical Site Characterization – May 2002)

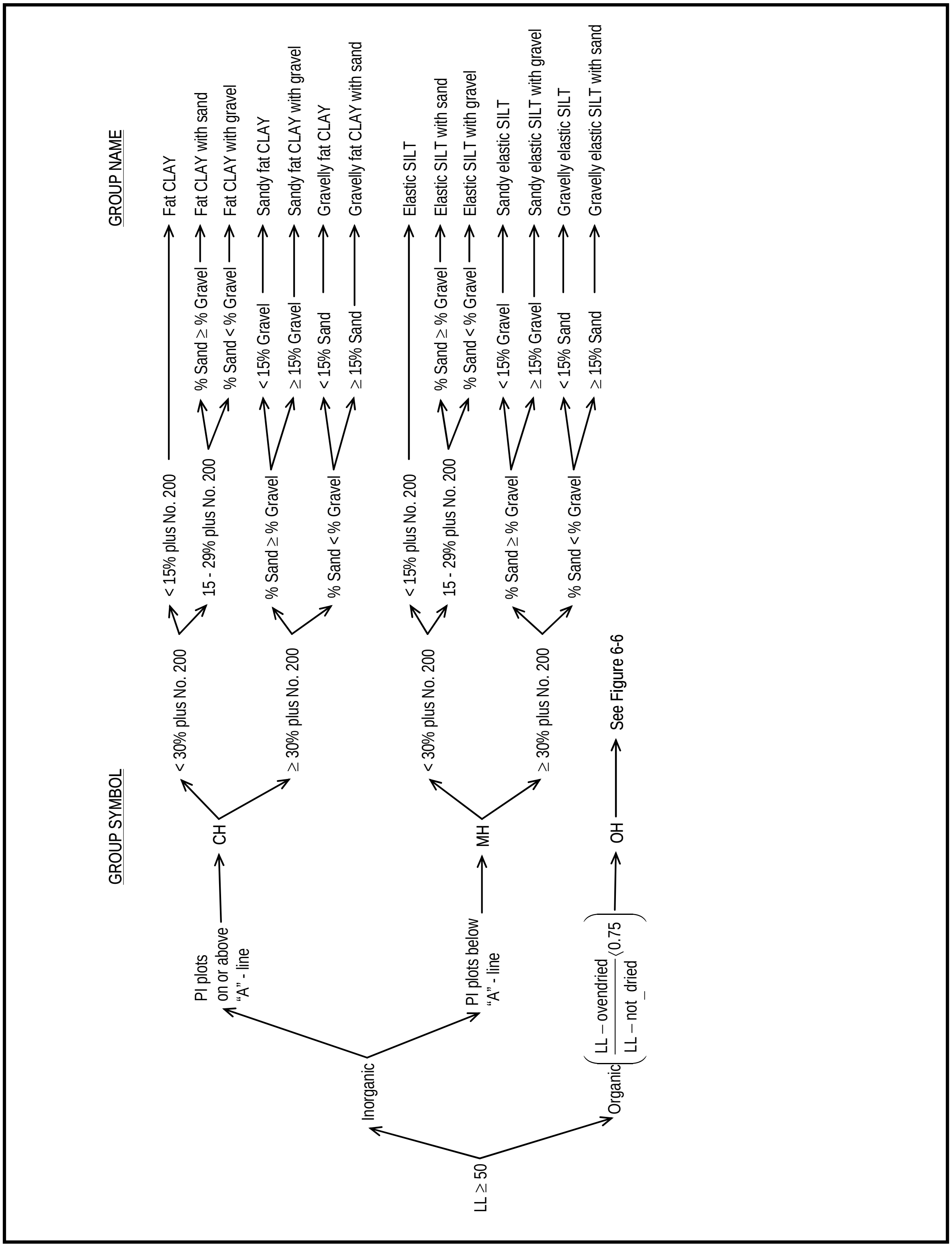


Figure 6-4, Group Symbol and Group Name for Fine-Grained Soils (LL ≥ 50)
(Subsurface Investigations – Geotechnical Site Characterization – May 2002)

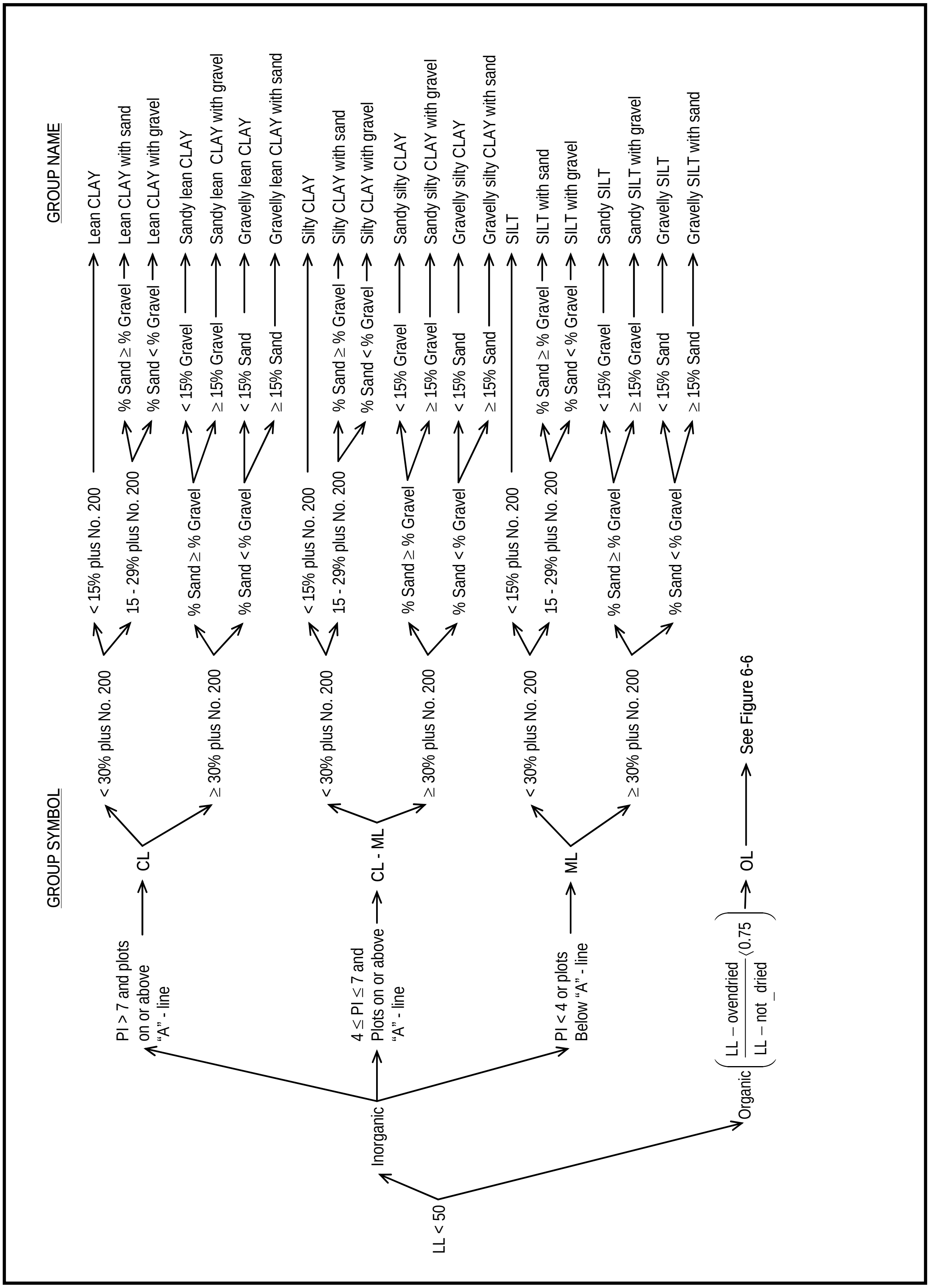


Figure 6-5, Group Symbol and Group Name for Fine-Grained Soils (LL < 50) (Subsurface Investigations – Geotechnical Site Characterization – May 2002)

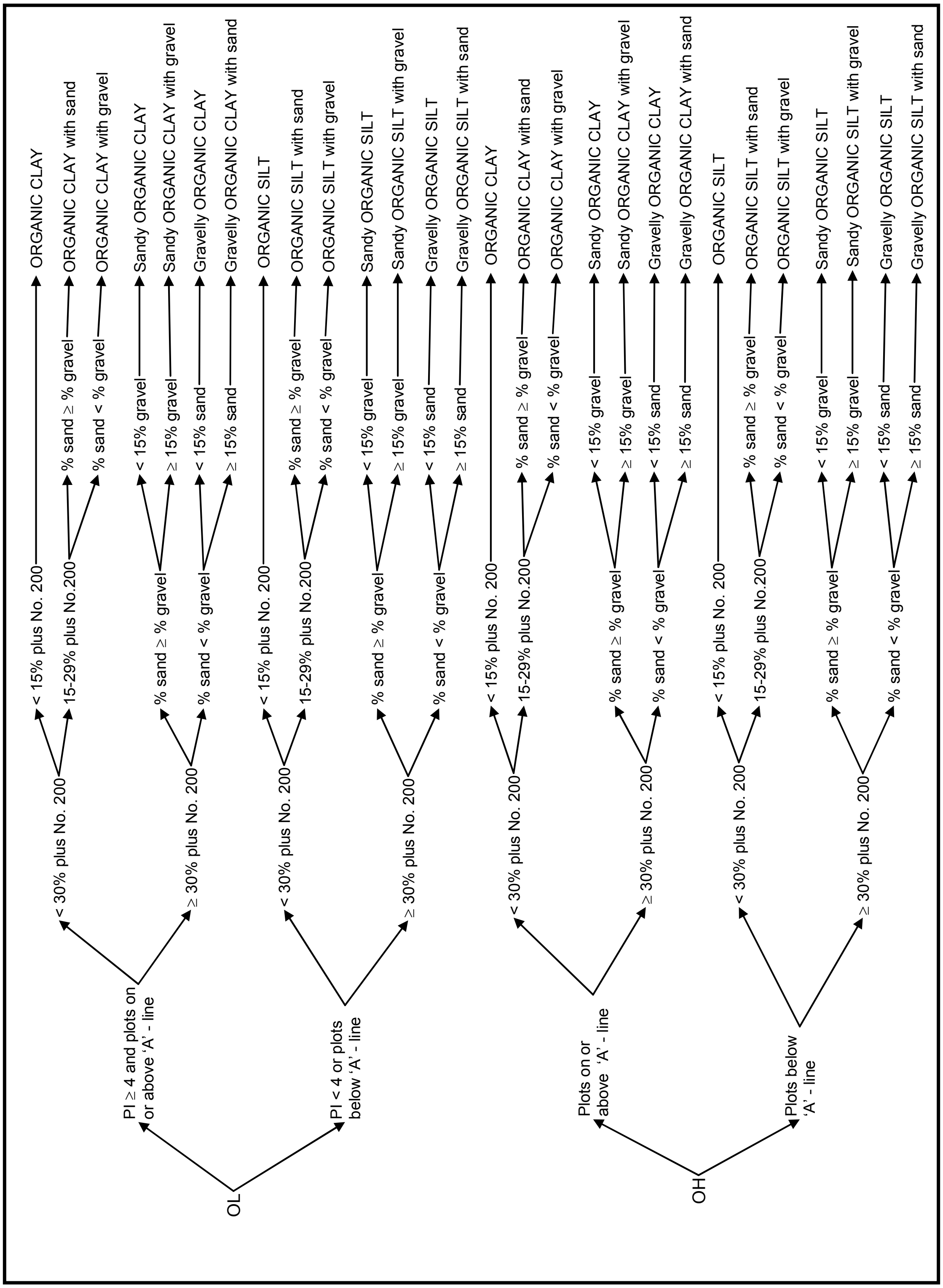


Figure 6-6, Group Symbol and Group Name for Organic Soils (Subsurface Investigations – Geotechnical Site Characterization – May 2002)

6.2.9 AASHTO Soil Classification System (AASHTO)

Terzaghi and Hogentogler originally developed this classification system for the U.S. Bureau of Public Roads in the late 1920s. This classification system divides all soils into eight major groups designated A-1 through A-8 (see Figures 6-7 and 6-8). In this classification system, the lower the number the better the soil is for subgrade materials. Coarse-grained soils are defined by groups A-1 through A-3, while groups A-4 through A-7 define the fine-grained soils. Group A-4 and A-5 are predominantly silty soils and group A-6 and A-7 are predominantly clayey soils. Group A-8 refers to peat and muck soils.

Groups A-1 through A-3 have 35 percent or less passing the No. 200 sieve, while groups A-4 through A-7 have more than 35 percent passing the No. 200 sieve. The classification system is presented in Figure 6-8. Table 6-10 indicates the gradation requirements used in the AASHTO classification system.

Table 6-10, AASHTO Gradation Requirements

Soil Component	Grain Size
Gravel	between 3" to No. 10
Sand	between No. 12 to No. 200
Silt and Clay	less than No. 200

For soils in Groups A-2, A-4, A-5, A-6 and A-7 the plasticity of the fines is defined in Table 6-11.

Table 6-11, AASHTO Plasticity Requirements

Soil Component	Plasticity Index
Silty	≤ 10%
Clayey	≥ 11%

To evaluate the quality of a soil as a highway subgrade material, a number called the Group Index (GI) is incorporated with the groups and subgroups of the soil. The GI is written in parenthesis after the group or subgroup designation. The GI is determined by the following equation:

$$\mathbf{GI} = (\mathbf{F} - \mathbf{35})[\mathbf{0.2} + \mathbf{0.005}(\mathbf{LL} - \mathbf{40})] + \mathbf{0.01}(\mathbf{F} - \mathbf{15})(\mathbf{PI} - \mathbf{10})$$ **Equation 6-3**

Where:

F = percent passing No. 200 sieve (in percent)
LL = Liquid Limit
PI = Plasticity Index

Listed below are some rules for determining the GI:

If the equation yields a negative value for the GI, use zero;
Round the GI to the nearest whole number, using proper rules of rounding;
There is no upper limit to the GI;
These groups, A-1-a, A-1-b, A-2-4, A-2-5, A-3, will always have a GI of zero;
The GI for groups A-2-6 and A-2-7 is calculated using the following equation

$$\mathbf{GI} = \mathbf{0.01}(\mathbf{F} - \mathbf{15})(\mathbf{PI} - \mathbf{10})$$ **Equation 6-4**

Figure 6-7 provides the range of liquid limit and plasticity index for group A-2 to A-7 soils.

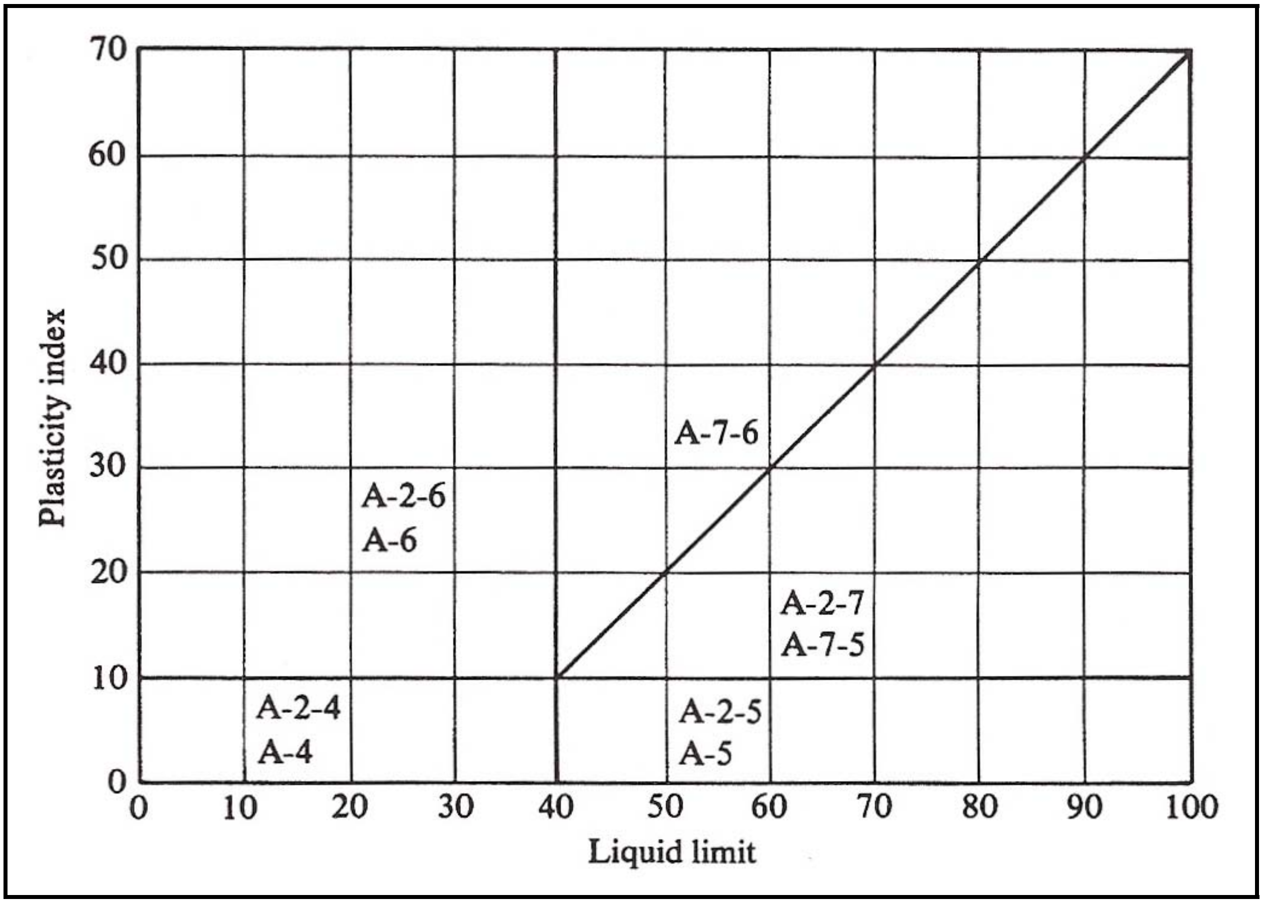


Figure 6-7, Range of LL and PI for Soils in Groups A-2 through A-7
(Subsurface Investigations – Geotechnical Site Characterization – May 2002)

GENERAL CLASSIFICATION	GRANULAR MATERIALS (35 percent or less of total sample passing No. 200)							SILT-CLAY MATERIALS (More than 35 percent of total sample passing No. 200)			
GROUP CLASSIFICATION	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5, A-7-6
Sieve analysis, percent passing:											
2 mm (No. 10)	50 max.										
0.425 mm (No. 40)	30 max.	50 max.	51 min.								
0.075 mm (No. 200)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid limit				40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.*
Usual significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
Group Index**	0		0	0		4 max.		8 max.	12 max.	16 max.	20 max.

Classification procedure: With required test data available, proceed from left to right on chart; correct group will be found by process of elimination. The first group from left into which the test data will fit is the correct classification.

*Plasticity Index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity Index of A-7-6 subgroup is greater than LL minus 30 (see Fig 4-9).

**See group index formula (Eq. 4-1) Group index should be shown in parentheses after group symbol as: A-2-6(3), A-4(5), A-6(12), A-7-5(17), etc.

Figure 6-8, AASHTO Soil Classification System
(Subsurface Investigations – Geotechnical Site Characterization – May 2002)

6.2.10 Other Pertinent Information

Additional information may also be included that adds to the description of the soil. This may include the geologic formation to which the soil belongs. This information should enhance to the description.

6.3 ROCK DESCRIPTION AND CLASSIFICATION

Rock descriptions should use technically correct geologic terms, although accepted local terminology may be used provided the terminology helps to describe distinctive characteristics. Rock cores should be logged when wet for consistency of color description and greater visibility of rock features. Geologists classify all rocks according to their origin and into three distinctive types as indicated in Table 6-12. All three rock types are found here in South Carolina: igneous rocks are found in the Piedmont region, metamorphic rocks are found in the Piedmont and Blue Ridge regions, and sedimentary rocks are found in the Coastal Plain. The Department uses both the geological history as well as the engineering properties to describe rock materials.

Table 6-12, Rock Classifications

Rock Type	Definition
Igneous	Derived from molten material
Sedimentary	Derived from settling, depositional, or precipitation processes
Metamorphic	Derived from pre existing rocks due to heat, fluids, and/or pressure.

The geologic conditions of South Carolina have a direct bearing on the activities of SCDOT. This is because the geological history of a rock will determine its mechanical behavior. Therefore, construction costs for a project, especially a new project with substantial foundation construction, are frequently driven by geological, subsurface factors. It is for this reason that much of the initial site investigation for a project requiring foundation work focuses on mechanical behavior of the subsurface materials within the construction limits. A detailed geologic description shall include the following items, in order:

1. Rock Type
2. Color
3. Grain-Size and shape
4. Texture (stratification/foliation)
5. Mineral Composition
6. Weathering and alteration
7. Strength
8. Rock Discontinuity
9. Rock Fracture Description
10. Other pertinent information

Rock Quality Designation (RQD) is used to indicate the quality of the rock and is frequently accompanied with descriptive words. It is always expressed as a percent. Percent recovery can be greater than 100 percent if the core from a subsequent run is recovered during a later run. Figure 6-9 further illustrates the determination of the RQD.

6.3.1 Igneous

Intrusive, or plutonic, igneous rocks have coarse-grained (large, intergrown crystals) texture and are believed to have been formed below the earth's surface. Granite and gabbro are examples of intrusive igneous rocks found in South Carolina. Extrusive, or volcanic, igneous rocks have fine-grained (small crystals) texture and have been observed to form at or above the earth's surface. Basalt and tuff are examples of an extrusive igneous rocks found in South Carolina. Pyroclastic igneous rocks are the result of a volcanic eruption and the rapid cooling of lava, examples of this type of rock are pumice and obsidian. Pyroclastic igneous rocks are not native to South Carolina.

6.3.2 Sedimentary

Sedimentary rocks are the most common form of rock and are the result of weathering of other rocks and the deposition of the rock sediment and soil. Sedimentary rocks are classified into three groups called clastic, chemical, and organic. Clastic rocks are composed of sediment (from weathering of rock or erosion of soil). Mudstone and sandstone are examples of clastic sedimentary rock found in South Carolina. Chemical sedimentary rocks are formed from materials carried in solution into lakes and seas. Limestone, dolomite, and halite are examples of this type of sedimentary rock. Organic sedimentary rocks are formed from the decay and deposition of organic materials in relatively shallow water bodies. Examples of organic sedimentary rocks are chalk, shale, coal, and coquina. Coquina is found within South Carolina.

6.3.3 Metamorphic

Metamorphic rocks result from the addition of heat, fluid, and/or pressure applied to preexisting rocks. This rock is normally classified into three types, strongly foliated, weakly foliated, and nonfoliated. Foliation refers to the parallel, layered minerals orientation observed in the rock. Schist is an example of a strongly foliated rock. Gneiss (pronounced "nice") is an example of a weakly foliated rock, while marble is an example of a nonfoliated rock. Schist, gneiss, slate and marble are metamorphic rocks found in South Carolina.

6.3.4 Rock Type

The rock type will be identified by either a licensed geologist or geotechnical engineer. Rocks are classified according to origin into the three major groups, which are igneous, sedimentary and metamorphic. These groups are subdivided into types based on mineral and chemical composition, texture, and internal structure.

6.3.5 Rock Color

The color of the rock shall be determined using the Munsell Color Chart and shall be described while the rock is still at or near the in-situ moisture condition. The color designation shall be provided at the end of the rock description.

6.3.6 Grain Size and Shape

Grain size is dependent on the type of rock as described previously; sedimentary rocks will have a different grain size and shape, when compared to igneous rocks. Metamorphic rocks may or may not display relict grain size of the original parent rock. The grain size description should be classified using the terms presented in Table 6-13. Angularity is a geologic property of particles and is also used in rock classification. Table 6-14 shows the grain shape terms and characteristics used for sedimentary rocks.

Table 6-13, Grain Size Terms for Sedimentary Rocks

Description	Diameter (mm)	Characteristic
Very coarse grained	>4.75	Grain sizes greater than popcorn kernels
Coarse grained	2.00 – 4.75	Individual grains easy to distinguish by eye
Medium grained	0.425 – 2.00	Individual grains distinguished by eye
Fine grained	0.075 – 0.425	Individual grains distinguished with difficulty
Very fine grained	<0.075	Individual grains cannot be distinguished by unaided eye

Table 6-14, Grain Shape Terms for Sedimentary Rocks

Description	Characteristic
Angular	Shows little wear, edges and corners are sharp, secondary corners are numerous and sharp
Subangular	Shows definite effects of wear; edges and corners are slightly rounded off; secondary corners are less numerous and less sharp than angular grains
Subrounded	Shows considerable wear; edges and corners are rounded to smooth curves; secondary corners greatly reduced and highly rounded
Rounded	Shows extreme wear; edges and corners smoother to broad curves; secondary corners are few and rounded
Well - Rounded	Completely worn; edges and corners are not present; no secondary edges

6.3.7 Texture (stratification/foliation)

Significant nonfracture structural features should be described. Stratification refers to the layering effects within sedimentary rocks, while foliation refers to the layering within metamorphic rocks. The thickness of the layering should be described using the terms of Table 6-15. The orientation of the bedding (layering)/foliation should be measured from the horizontal with a protractor.

Table 6-15, Stratification/Foliation Thickness Terms

Descriptive Term	Layer Thickness
Very Thickly Bedded	>1.0 m
Thickly Bedded	0.5 to 1.0 m
Thinly Bedded	50 to 500 mm
Very Thinly Bedded	10 to 50 mm
Laminated	2.5 to 10 mm
Thinly Laminated	<2.5 mm

6.3.8 Mineral Composition

The mineral composition should be identified by a geologist or geotechnical engineer based on experience and the use of appropriate references. The most abundant mineral should be listed first, followed by minerals in decreasing order of abundance. For some common rock types, mineral composition need not be specified (e.g. dolomite and limestone).

6.3.9 Weathering and Alteration

Weathering as defined here is due to physical disintegration of the minerals in the rock by atmospheric processes while alteration is defined here as due to geothermal processes.

Table 6-16, Weathering/Alteration Terms

Description	Recognition
Residual Soil	Original minerals of rock have been entirely decomposed to secondary minerals, and original rock fabric is not apparent; material can be easily broken by hand
Completely Weathered / Altered	Original minerals of rock have been almost entirely decomposed to secondary minerals, although the original fabric may be intact; material can be granulated by hand
Highly Weathered / Altered	More than half of the rock is decomposed; rock is weakened so that a minimum 1-7/8 inch diameter sample can be easily broken readily by hand across rock fabric
Moderately Weathered / Altered	Rock is discolored and noticeably weakened, but less than half is decomposed; a minimum 1-7/8 inch diameter sample cannot be broken readily by hand across rock fabric
Slightly Weathered / Altered	Rock is slightly discolored, but not noticeably lower in strength than fresh rock
Fresh	Rock shows no discoloration, loss of strength, or other effect of weathering / alteration

6.3.10 Strength

Table 6-17 presents guidelines for common qualitative assessment of strength while mapping or during primary logging of rock cores at the site by using a geologic hammer and pocketknife. The field estimates should be confirmed where appropriate by comparisons with selected laboratory test.

Table 6-17, Rock Strength Terms

Description	Recognition	Approximate Uniaxial Compressive Strength (psi)
Extremely Weak Rock	Can be indented by thumbnail	35 – 150
Very Weak Rock	Can be peeled by pocket knife	150 –700
Weak Rock	Can be peeled with difficulty by pocket knife	700 – 3,500
Medium Strong Rock	Can be indented 3/16 inch with sharp end of pick	3,500 – 7,200
Strong Rock	Requires one hammer blow to fracture	7,200 – 14,500
Very Strong Rock	Requires many hammer blows to fracture	14,500 – 35,000
Extremely Strong Rock	Can only be chipped with hammer blows	> 35,000

A popular classification system based on quantifying discontinuity spacing is known as the RQD. RQD is illustrated in Table 6-18 and is defined as the total combined length of all the pieces of the intact core that are longer than twice the diameter of the core (normally 2 inches) recovered during the core run divided by the total length of the core run (i.e. the summation of rock pieces greater than 4 inches in length is 4 feet for a 5-foot run indicating an RQD of 80 percent).

Table 6-18, Rock Quality Description Terms

Description	RQD
Very poor	0 - 25%
Poor	26% - 50%
Fair	51% - 75%
Good	76% - 90%
Excellent	91% - 100%

The scratch hardness test can also be used to provide an indication of the hardness of a rock sample. The terms to describe rock hardness are provided in Table 6-19.

Table 6-19, Rock Hardness Terms

Description	Characteristic
Soft (S)	Plastic materials only
Friable (F)	Easily crumbled by hand, pulverized or reduced to powder
Low Hardness (LH)	Can be gouged deeply or carved with a pocketknife
Moderately Hard (MH)	Can be readily scratched by a knife blade
Hard (H)	Can be scratched with difficulty
Very Hard (VH)	Cannot be scratched by pocketknife

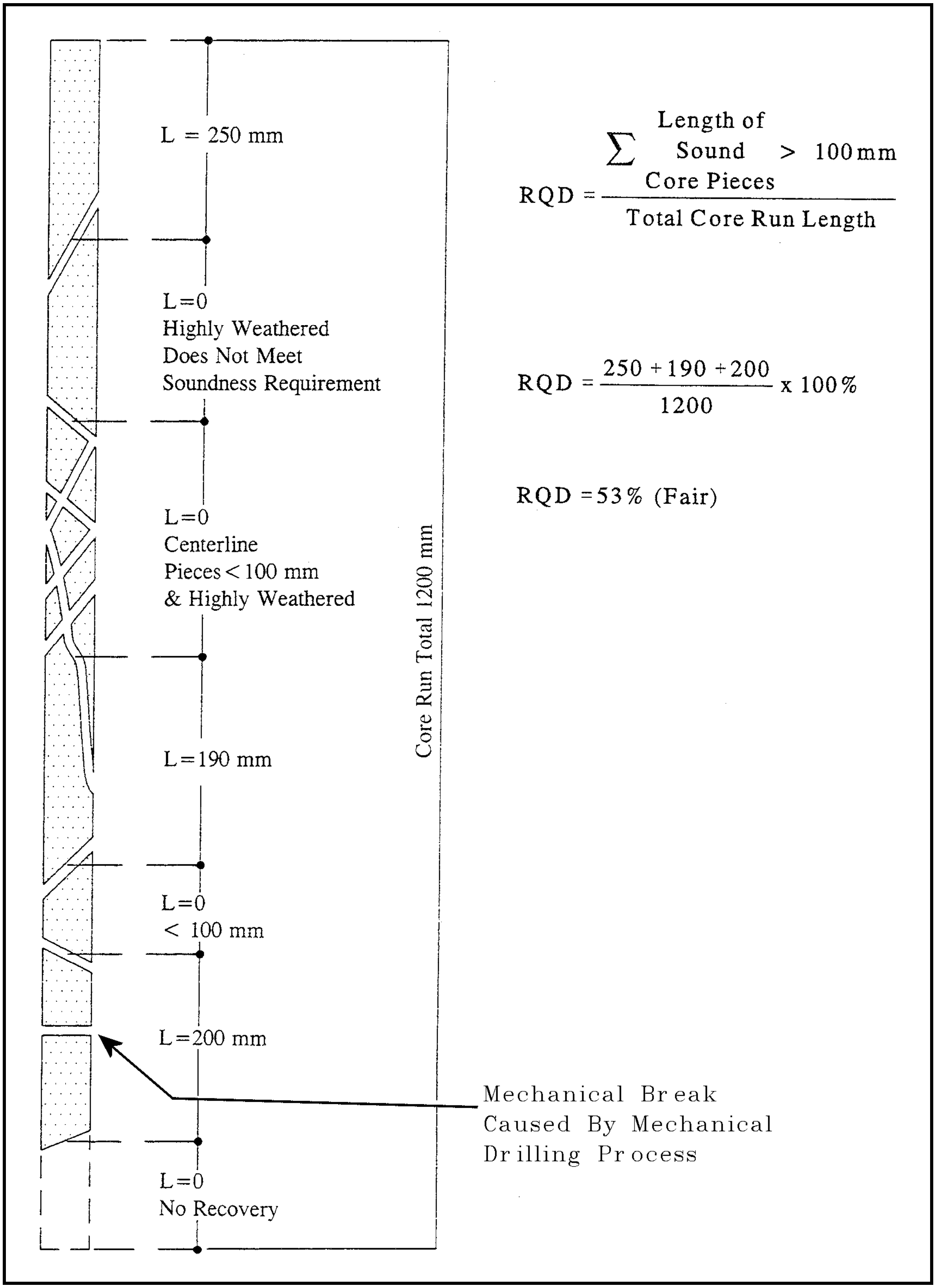


Figure 6-9, RQD Determination
(Subsurface Investigations – Geotechnical Site Characterization – May 2002)

6.3.11 Rock Discontinuity

Discontinuity is the general term for any mechanical crack or fissure in a rock mass having zero or low tensile strength. It is the collective term for most types of joints, weak bedding planes, weak schistosity planes, weakness zones, and faults. The symbols recommended for the type of rock mass discontinuities are listed in Table 6-20.

Table 6-20, Discontinuity Type

Symbol	Description
F	Fault
J	Joint
Sh	Shear
Fo	Foliation
V	Vein
B	Bedding

The spacing of discontinuities is the perpendicular distance between adjacent discontinuities. The spacing is measured in feet, perpendicular to the planes in the set. Table 6-21 presents guidelines to describe discontinuity.

Table 6-21, Discontinuity Spacing

Symbol	Description
EW	Extremely Wide (>65 feet)
W	Wide (22 – 65 feet)
M	Moderate (7.5 – 22 feet)
C	Close (2 – 7.5 feet)
VC	Very Close (<2 feet)

The discontinuities should be described as closed, open, or filled. Aperture is used to describe the perpendicular distance separating the adjacent rock walls of an open discontinuity in which the intervening space is air or water filled. Width is used to describe the distance separating the adjacent rock walls of filled discontinuities. The terms presented in Table 6-22 and Table 6-23 should be used to describe apertures and widths, respectively. Terms such as “wide”, “narrow”, and “tight” are used to describe the width of discontinuities such as thickness of veins, fault gouge filling, or joint openings. For the faults or shears that are not thick enough to be represented on the soil test boring log, the measured thickness is recorded numerically in millimeters (mm).

Table 6-22, Aperture Size Discontinuity Terms

Aperture Opening	Description	
<0.1 mm	Very tight	Closed Features
0.1 – 0.25 mm	Tight	
0.25 – 0.5 mm	Partly open	
0.5 – 2.5 mm	Open	Gapped Features
2.5 – 10 mm	Moderately open	
>10 mm	Wide	
1 – 10 cm	Very wide	Open Features
10 – 100 cm	Extremely wide	
>1m	Cavernous	

Table 6-23, Discontinuity Width Terms

Symbol	Description
W	Wide (12.5 – 50 mm)
MW	Moderately Wide (2.5 – 12.5 mm)
N	Narrow (1.25 – 2.5 mm)
VN	Very Narrow (<1.25 mm)
T	Tight (0 mm)

In addition to the above characterizations, discontinuities are further characterized by the surface shape of the joint and the roughness of its surface (see Table 6-24 and 6-25).

Table 6-24, Surface Shape of Joint Terms

Symbol	Description
Wa	Wavy
Pl	Planar
St	Stepped
Ir	Irregular

Table 6-25, Surface Roughness Terms

Symbol	Description
Slk	Slickensided (surface has smooth, glassy finish with visual evidence of striations)
S	Smooth (surface appears smooth and feels so to the touch)
SR	Slightly Rough (asperities on the discontinuity surfaces are distinguishable and can be felt)
R	Rough (some ridges and side-angle steps are evident; asperities are clearly visible, and discontinuity surface feels very abrasive)
VR	Very Rough (near-vertical steps and ridges occur on the discontinuity surface)

Filling is the term for material separating the adjacent rock walls of discontinuities. Filling is characterized by its type, amount, width (i.e. perpendicular distance between adjacent rock walls (See Table 6-23)), and strength. Table 6-26 presents guidelines for characterizing the amount of filling.

Table 6-26, Filling Amount Terms

Symbol	Description
Su	Surface Stain
Sp	Spotty
Pa	Partially Filled
Fi	Filled
No	None

6.3.12 Rock Fracture Description

The location of each naturally occurring fracture and mechanical break should be shown in the fracture column of the rock core log. The naturally occurring fractures are numbered and described using the terminology described above for discontinuities.

The naturally occurring fractures and mechanical breaks are sketched in the drawing column of the Soil Test Boring Log (see Figure 6-10). Dip angles of fractures should be measured using a protractor and marked on each log. If the rock is broken into many pieces less than 1 inch long, the log may be crosshatched in that interval or the fracture may be shown schematically.

The number of naturally occurring fractures observed in each 1 foot of core should be recorded in the fracture frequency column. Mechanical breaks, thought to have occurred due to drilling, are not counted. The following criteria can be used to identify natural breaks:

1. A rough brittle surface with fresh cleavage planes in individual rock minerals indicates an artificial fracture.

2. A generally smooth or somewhat weathered surface with soft coating or infilling materials, such as talc, gypsum, chlorite, mica, or calcite obviously indicates a natural discontinuity.

3. In rocks showing foliation, cleavage, or bedding it may be difficult to distinguish between natural discontinuities and artificial fractures when these are parallel with the incipient weakness planes. If drilling has been carried out carefully, then the questionable breaks should be counted as natural features, to be on the conservative side.

4. Depending upon the drilling equipment, part of the length of core being drilled may occasionally rotate with the inner barrels in such a way that grinding of the surfaces of discontinuities and fractures occur. In weak rock types, it may be very difficult to decide if the resulting rounded surfaces represent natural or artificial features. When in doubt, the conservative assumption should be made; i.e. assume that the discontinuities are natural.

The results of core logging (frequency and RQD) can be strongly time dependent and moisture content dependent in case of certain varieties of shales and mudstones having relatively weakly developed diagenetic bonds. A frequent problem is “discing”, in which an initially intact core separates into discs on incipient planes, the process becoming noticeable perhaps within minutes of core recovery. This phenomenon is experienced in several different forms:

1. Stress relief cracking (and swelling) by the initially rapid release of strain energy in cores recovered from areas of high stress, especially in the case of shaley rocks.

2. Dehydration cracking experienced in the weaker mudstones and shales which may reduce RQD from 100 percent to 0 percent in a matter of minutes, the initial integrity possibly being due to negative pore pressure.

3. Slaking cracking experienced by some of the weaker mudstones and shales when subjected to wetting and drying.

All these phenomena may make core logging of fracture frequency and RQD unreliable. Whenever such conditions are anticipated, core should be logged by an experienced geologist or geotechnical engineer as it is recovered and at subsequent intervals when the phenomenon is predicted. An added advantage is that mechanical index tests, such as point load index or Schmidt hammer, while the core is still in a saturated state.

6.3.13 Other Pertinent Information

Additional information may also be included that adds to the description of the soil. This may include the geologic formation to which the soil belongs. This information should enhance to the description.

6.3.14 Rock Mass Rating

The information obtained in the preceding sections is used to develop the Rock Mass Rating (RMR). The RMR is used to determine how the mass of rock will behave as opposed to the samples used in unconfined compression, which typically tend to represent the firmest materials available. Discontinuities effect the ability of rock to carry load and to resist deformations. The RMR is the sum of the relative ratings (RR) for 5 parameters adjusted for joint orientations. Table 6-27 provides the 5 parameters and the range of values. The RMR is adjusted to account for joint orientation depending on the favorability of the joint orientation for the specific project. Table 6-28 contains the rating adjustments (RRA) for joint orientation. The adjusted RMR is determined using Equation 6-5. The description of the rock mass is based on the adjusted RMR as defined in Table 6-29. The adjusted RMR can be used to estimate the rock mass shear strength and the deformation modulus (see Chapter 7).

$$\mathbf{RMR} = \mathbf{RR1} + \mathbf{RR2} + \mathbf{RR3} + \mathbf{RR4} + \mathbf{RR5} + \mathbf{RRA} \qquad \textbf{Equation 6-5}$$

Table 6-27, Classification of Rock Masses

	Parameter		Range of Values						
1	Strength of intact rock material	Point load strength index	>1,215 psi	1,215 – 1,100 psi	300 – 1,100 psi	150 – 300 psi	For this low range, uniaxial compressive test is perferred		
		Uniaxial compressive strength	>30,000 psi	30,000 – 15,000 psi	7,500 – 15,000 psi	3,600 – 7,500 psi	1,500 – 3,600 psi	500 – 1,500 psi	150 – 500 psi
	Relative Rating (RR1)		15	12	7	4	2	1	0
2	Drill core quality RQD		90 – 100%	75 – 90%	50 – 75%	25 – 50%	<25%		
	Relative Rating (RR2)		20	17	13	8	3		
3	Spacing of Joints		>10 ft	3 – 10 ft	1 – 3 ft	2 in – 1 ft	<2 in		
	Relative Rating (RR3)		30	25	20	10	5		
4	Condition of Joints		- Very rough surfaces - Not continuous - No separation - Hard joint wall rock	- Slightly rough surfaces - Separation <0.05 in - Hard joint wall rock	- Slightly rough surfaces - Separation <0.05 in - Soft joint wall rock	- Slicken-sided surfaces or - Gouge <0.2 in thick or - Joints open 0.05 – 0.2 in - Continuous joints	- Soft gouge >0.2 in thick or - Joints open >0.2 in - Continuous joints		
	Relative Rating (RR4)		25	20	12	6	0		
5	Ground water conditions	Ratio – joint water pressure/major principal stress	0	0.0 – 0.2	0.2 – 0.5	>0.5			
		General conditions	Completely dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems			
	Relative Rating (RR5)		10	7	4	0			

Table 6-28, Rating Adjustment for Joint Orientations

Strike and Dip Orientations of Joints		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Relative Ratings (RRA)	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

Table 6-29, Rock Mass Class Determination

RMR Rating	81 – 100	61 – 80	41 – 60	21 – 40	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

6.4 BORING RECORDS

Field logs, for soil test borings, shall be prepared by the driller at the time of drilling, while a licensed geologist or geotechnical engineer shall prepare the field logs for rock coring. The field logs shall be reviewed by an experienced geotechnical engineer or geologist. In addition, the geotechnical engineer/geologist shall also review all samples to confirm the accuracy of the field logs. Preliminary Soil Test Boring Logs shall be prepared and forwarded to the geotechnical designer for selection of samples for laboratory testing. At the completion of laboratory testing, the preliminary logs shall be corrected to conform to the results of the laboratory testing and final Soil Test Boring Logs shall be prepared and submitted. Figure 6-10 provides the log for use on SCDOT projects. Figures 6-11 and 6-12 provide the descriptors to be used in preparing the logs.

SCDOT Soil Test Boring Log

File No.:	727.615	**Project No. (PIN):**	23546	**County:**	Beaufort/Jasper	**Eng./Geo.:**	A. Bore
Site Description:	RBO New River					**Route:**	SC 170/46
Boring No.:	B-722	**Boring Location:**	722+00	**Offset:**	5 ft LT	**Alignment:**	Mainline
Elev.:	1,500 ft	**Latitude:**	34.3750	**Longitude:**	81.0944	**Date Started:**	07/15/03
Total Depth:	45 ft	**Soil Depth:**	39 ft	**Core Depth:**	6 ft	**Date Completed:**	07/16/03
Bore Hole Diameter (in):	4.5	**Sampler Configuration**		**Liner required:**	Y N	**Liner used:**	Y N
Drill Machine:	CME-750	**Drill Method:**	Wash Rotary	**Hammer Type:**	Automatic	**Energy Ratio:**	100%
Core Size:	NQ Wireline	**Driller:**	I. Core	**Groundwater:**	**TOB** 7.5 ft	**24 hr**	15 ft

Depth (feet)	Elevation (ft msl)	MATERIAL DESCRIPTION	Graphic Log	Sample Depth (feet)	Sample Type / No.	1 st	2 nd	3 rd	SPT N-Value	● - SPT N-Value (blows / foot) PL MC LL X-------O-------X ▲ - % fines 10 20 30 40 50 60 70 80 90 100
		Soil Description a , b , c , d , e , f , g h , i , j , Munsell , LL PL , PI , NMC , %#200 Munsell = Munsell Color Chart Designation LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index NMC = Natural Moisture Content %#200 = Percent Passing #200 Sieve								
		Rock Description (as required) Lithologic description: rock type, color, texture, grain size, foliation, weathering and strength with k , l , m , n , o , p , q r , Munsell , RQD , %REC RMR Munsell = Munsell Color Chart Designation RQD = Rock Quality Designation %REC = Percent Recovery RMR = Rock Mass Rating								

Figure 6-10, SCDOT Soil Test Boring Log

SCDOT Soil Test Boring Log Descriptors

a - Relative Density / Consistency Terms

Relative Density[1]

Descriptive Term	Relative Density	SPT Blow Count
Very Loose	0 to 15%	< 4
Loose	16 to 35%	5 to 10
Medium Dense	36 to 65%	11 to 30
Dense	66 to 85%	31 to 50
Very Dense	86to 100%	>51

Consistency[2]

Descriptive Term	Unconfined Compression Strength (q_u) (tsf)	SPT Blow Count
Very Soft	<0.25	<2
Soft	0.26 to 0.50	3 to 4
Firm	0.51 to 1.00	5 to 8
Stiff	1.01 to 2.00	9 to 15
Very Stiff	2.01 to 4.00	16 to 30
Hard	>4.01	> 31

b Moisture Condition

Descriptive Term	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually in coarse-grained soils below the water table

c Color

Describe the sample color while sample is still moist, using Munsell color chart.

d Angularity[1]

Descriptive Term	Criteria
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges

e HCl Reaction[3]

Descriptive Term	Criteria
None Reactive	No visible reaction
Weakly Reactive	Some reaction, with bubbles forming slowly
Strongly Reactive	Violent reaction, with bubbles forming immediately

f Cementation[3]

Descriptive Term	Criteria
Weakly Cemented	Crumbles or breaks with handling or little finger pressure
Moderately Cemented	Crumbles or breaks with considerable finger pressure
Strongly Cemented	Will not crumble or break with finger pressure

g Particle-Size Range[1]

Gravel

	mm	Sieve size
Fine	4.76 to 19.1	#4 to ¾ inch
Coarse	19.1 to 76.2	¾ inch to 3 inch

Sand

	mm	Sieve size
Fine	0.074 to 0.42	#200 to #40
Medium	0.42 to 2.00	#40 to #10
Coarse	4.00 to 4.76	#10 to #4

h Primary Soil Type[1, 2]

The primary soil type will be shown in all capital letters

i USCS Soil Designation

Indicate USCS soil designation as defined in ASTM D-2487 and D-2488

j AASHTO Soil Designation

Indicate AASHTO soil designation as defined in AASHTO M-145 and ASTM D-3282

[1]Applies to coarse-grained soils (major portion retained on No. 200 sieve)
[2]Appiles to fine-grained soils (major portion passing No. 200 sieve)
[3]Use as required

Figure 6-11, SCDOT Soil Test Boring Log Descriptors - Soil

SCDOT Soil Test Boring Log Descriptors

ROCK WEATHERING / ALTERATION

Description	Recognition
Residual Soil	Original minerals of rock have been entirely decomposed to secondary minerals, and original rock fabric is not apparent; material can be easily broken by hand
Completely Weathered / Altered	Original minerals of rock have been almost entirely decomposed to secondary minerals, although the original fabric may be intact; material can be granulated by hand
Highly Weathered / Altered	More than half of the rock is decomposed; rock is weakened so that a minimum 1-7/8 inch diameter sample can be easily broken readily by hand across rock fabric
Moderately Weathered / Altered	Rock is discolored and noticeably weakened, but less than half is decomposed; a minimum 1-7/8 inch diameter sample cannot be broken readily by hand across rock fabric
Slightly Weathered / Altered	Rock is slightly discolored, but not noticeably lower in strength than fresh rock
Fresh	Rock shows no discoloration, loss of strength, or other effect of weathering / alteration

ROCK STRENGTH

Description	Recognition	Approximately Uniaxial Compressive Strength (psi)
Extremely Weak Rock	Can be indented by thumbnail	35 – 150
Very Weak Rock	Can be peeled by pocket knife	150 –700
Weak Rock	Can be peeled with difficulty by pocket knife	700 – 3,500
Medium Strong Rock	Can be indented 3/16 inch with sharp end of pick	3,500 – 7,200
Strong Rock	Requires one hammer blow to fracture	7,200 – 14,500
Very Strong Rock	Requires many hammer blows to fracture	14,500 – 35,000
Extremely Strong Rock	Can only be chipped with hammer blows	> 35,000

DISCONTINUITY DESCRIPTORS

k - Dip of fracture surface measured relative to horizontal with bearing and direction

l Discontinuity Type
- F - Fault
- J - Joint
- Sh - Shear
- Fo - Foliation
- V - Vein
- B - Bedding

m Discontinuity Width (millimeters)
- W - Wide (12.5 – 50)
- MW - Moderately Wide (2.5 – 12.5)
- N - Narrow (1.25 – 2.5)
- VN - Very Narrow (< 1.25)
- T - Tight (0)

n Amount of Infilling
- Su - Surface Stain
- Sp - Spotty
- Pa - Partially Filled
- Fi - Filled
- No - None

o Type of Infilling
- Cl - Clay
- Ca - Calcite
- Ch - Chloride
- Fe - Iron Oxide
- Gy - Gypsum/Talc
- H - Healed
- No - None
- Py - Pyrite
- Qz - Quartz
- Sd - Sand

p Surface Shape of Joint
- Wa - Wavy
- Pl - Planar
- St - Stepped
- Ir - Irregular

q Discontinuity Spacing (feet)
- EW Extremely Wide (> 65)
- W Wide (22 – 65)
- M Moderate (7.5 – 22)
- C Close (2 – 7.5)
- VC Very Close (< 2)

r Roughness of Surface
- Slk - Slickensided (surface has smooth, glassy finish with visual evidence of striations)
- S - Smooth (surface appears smooth and feels so to the touch)
- SR - Slightly Rough (asperities on the discontinuity surfaces are distinguishable and can be felt)
- R - Rough (some ridges and side-angle steps are evident; asperities are clearly visible, and discontinuity surface feels very abrasive
- VR - Very Rough (near-vertical steps and ridges occur on the discontinuity surface)

Figure 6-12, SCDOT Soil Test Boring Log Descriptors - Rock

6.5 REFERENCES

Department of Transportation, State of Florida (2006). Soils and Foundations Handbook. Gainesville, Florida, State Materials Office.

Federal Highway Administration, U.S. Department of Transportation (April 2002). Geotechnical Engineering Circular No. 5 – Evaluation of Soil and Rock Properties (Publication No. FHWA-IF-02-034). Washington, DC, U.S. Department of Transportation.

Federal Highway Administration, U.S. Department of Transportation (May 2002). Subsurface Investigations – Geotechnical Site Characterization (Publication No. FHWA NHI-01-031). Washington, DC, National Highway Institute.

Chapter 7

GEOMECHANICS

Final Draft

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Table of Contents

List of Tables

List of Figures

CHAPTER 7

GEOMECHANICS

7.1 INTRODUCTION

This chapter presents the geotechnical design philosophy of SCDOT. This philosophy includes the approach to the geotechnical investigations of the project, and the correlations that link the field and laboratory work that precedes this chapter to the engineering analysis that is subsequent to this chapter. The approach to the geotechnical investigation of transportation projects entails the use of preliminary and final explorations and reports. The development of an understanding of the regional and local geological environment and the effect of seismicity on the project is required. The geotechnical approach provided in this chapter is not meant to be the only approach, but a representative approach of the thought process expected to be used on SCDOT projects. The geotechnical engineer-of-record shall develop a design approach that reflects both the requirements of this Manual as well as a good standard-of-practice. While there is some flexibility in the approach to the design process, the correlations provided in this chapter must be used unless written permission is obtained in advance. All requests for changes shall be forwarded to the PCS/GDS for review prior to approval. These correlations were adopted after a review of the geotechnical state of practice within the United States.

7.2 GEOTECHNICAL DESIGN APPROACH

Geotechnical engineering requires the use of science, art, and economics to perform analyses and designs that are suitable for the public use. The science of geotechnical engineering consists of using the appropriate theories to interpret field data, develop geologic profiles, select foundation types, perform analyses, develop designs, plans and specifications, construction monitoring, maintenance, etc.

The art of geotechnical engineering is far more esoteric and relies on the judgment and experience of the engineer. This is accomplished by knowing applicability and limitations of the geotechnical analytical theories and assessing the uncertainties associated with soil properties, design methodologies, and the resulting impact on structural performance. The engineer is required to evaluate the design or analysis and decide if it is “reasonable” and will it meet the performance expectations that have been established. Reasonableness is a subjective term that depends on the engineer’s experience, both in design and construction. If the solution does not appear reasonable, the engineer should make the appropriate changes to develop a reasonable solution. In addition, the engineer should document why the first solution was not reasonable and why the second solution is reasonable. This documentation is an important part of the development of the design approach. If the solution appears reasonable, then design should proceed to the economics of geotechnical engineering.

The economics of geotechnical engineering assesses the effectiveness of the solution from a cost perspective. Sometimes geotechnical engineers get caught up in the science and art of geotechnical engineering and do not evaluate other non-geotechnical solutions that may be cost effective both in design and construction. For example, alternate alignments should be explored to avoid poor soils, decreasing vertical alignment to reduce surface loads, placing alternate designs on the plans to facilitate competitive bidding, etc. The science, art, and economics are not sequential facets of geotechnical engineering but are very often intermixed throughout the design process.

7.3 GEOTECHNICAL ENGINEERING QUALITY ASSURANCE

A formal internal geotechnical engineering quality assurance plan should be established for all phases of the geotechnical engineering process. The first-line geotechnical engineer is expected to perform analyses with due diligence and a self-prescribed set of checks and balances. The geotechnical quality control plan should include milestones in the project development where analysis, recommendations, etc. are reviewed by at least one other geotechnical engineer of equal experience or higher seniority. Formal documentation of the quality assurance process should be detectable upon review of geotechnical calculations, reports, etc. All engineering work shall be performed under the direct supervision of a Professional Engineer (P.E.) licensed by the South Carolina State Board of Registration for Professional Engineers and Surveyors in accordance with Chapter 22 of Title 40 of the 1976 Code of Laws of South Carolina, latest amendment.

7.4 DEVELOPMENT OF SUBSURFACE PROFILES

The SCDOT geotechnical design process indicated in Chapter 4, allows for a preliminary and a final geotechnical exploration program for all projects. The primary purpose of the preliminary exploration is to provide a first glance at the project, while the final exploration is to provide all of the necessary geotechnical information to complete the final design.

It is incumbent upon the geotechnical engineer to understand the geology of the project site and determine the potential effects of the geology on the project. The geotechnical engineer should also have knowledge of the regional geology that should be used in the development of the exploration program for the project. In addition to the geologic environment, the geotechnical engineer should be aware of the seismic environment (see Chapter 11 for geology and seismicity and Chapter 12 for site class discussions). The geotechnical engineer is also required to know and understand the impacts of the design earthquake event on the subsurface conditions at the project site (see Chapters 13 and 14 for the impacts and designs, respectively). The geologic formation and local seismicity may have a bearing on the selection of the foundation type and potential capacity. For example, for driven piles bearing in the Cooper Marl formation of the Charleston area, precast, prestressed concrete piles should penetrate the formation approximately 5 feet, with most of the capacity being developed by steel H-pile extensions attached below the prestressed pile, penetrating into the Marl.

The geotechnical engineer should develop a subsurface profile for both the preliminary and final geotechnical subsurface explorations. The subsurface profile developed should take into consideration the site variability as indicated in Section 7.5. The profile should account for all available data and is normally depicted along the longitudinal axis of the structure. However, in some cases, subsurface profiles transverse to the axis of the structure may be required to determine if a formation is varying (i.e. sloping bearing strata) along the transverse axis.

7.5 SITE VARIABILITY

Keeping in mind the geologic framework of the site, the geotechnical engineer should evaluate the site variability (SV). Site variation can be categorized as Low, Medium, or High. If a project site has a “High” site variability (SV), the extent of the “Site” should be subdivided to obtain smaller “Sites” with either Low or Medium variability. The use of a “High” site variability (SV) for geotechnical design shall only be allowed upon consultation with the PCS/GDS. The site variability (SV) determination may be based on judgment; however, justification for the selection of the site variability is required. Conversely, the determination of site variability may be based on the shear strength of the subsurface soils. The shear strength may be based on Standard Penetration Test (SPT), the Cone Penetration Test (CPT), or the results of other field or laboratory testing. Soil property (i.e. shear strength) selection for the determination of resistance factors and SV should be consistent with Chapter 9. If shear strengths are used to determine SV, then the Coefficient of Variation (COV) of the shear strengths shall be determined. The COV shall be used to determine the SV as indicated in Table 7-1.

Table 7-1, Site Variability Defined By Soil Shear Strength COV

Site Variability (SV)	COV
Low	< 25%
Medium	25% ≤ COV < 40%
High	≤ 40%

7.6 PRELIMINARY GEOTECHNICAL SUBSURFACE EXPLORATION

Prior to the commencement of the preliminary exploration, the geotechnical engineer shall visit the site and conduct a GeoScoping. The GeoScoping consists of the observation of the project site to identify areas that may impact the project from the geotechnical perspective. These areas may be selected for exploration during the preliminary exploration if the site is located within the existing SCDOT Right-of-Way (ROW). If the areas of concern are located outside of the existing SCDOT ROW, then the areas should be investigated during the final exploration. For projects conducted by SCDOT, the results of the GeoScoping shall be reported on the appropriate forms (see Appendix A). For consultant projects, the consultant shall use the form developed and approved by the consulting firm. The form shall be included as an appendix to the preliminary geotechnical report. An engineering professional with experience in observing and reviewing sites for potential geotechnical concerns shall be responsible for conducting the GeoScoping.

The preliminary exploration requirements are detailed in Chapter 4, while the contents of the preliminary geotechnical report are detailed in Chapter 21. The primary purpose of the preliminary exploration is to provide a first glance at the project. Typically the preliminary exploration will be short on project details. However, the most important details that will be known are what type of project is it (i.e. bridge replacement, new road, intersection improvement, etc.) and where the project is located. In many cases, the final alignment and structure locations may not be known. The primary purpose of this type of exploration is not to provide final designs, but to determine if there are any issues that could significantly affect the project. These issues should be identified and the potential impacts and consequences of these design issues evaluated. Design issues should be identified and documented for additional exploration during the final geotechnical exploration. If the project is located completely within the SCDOT ROW, then the entire exploration may be performed during the preliminary exploration phase of the project; however, the report prepared shall be a preliminary report that meets the requirements of Chapter 21.

7.7 FINAL GEOTECHNICAL SUBSURFACE EXPLORATION

The final geotechnical exploration shall conform to the requirements detailed in Chapter 4, while the contents of the final geotechnical report shall conform to the requirements detailed in Chapter 21. The final exploration shall be laid out to use the testing locations from the preliminary exploration to the greatest extent possible without compromising the results of the final exploration. The final exploration shall include those areas identified during the preliminary exploration or during the GeoScoping as requiring additional investigation. If these areas impact the performance of the project, these impacts shall be brought to the immediate attention of the Design/Program Manager. In addition, the geotechnical engineer shall also include recommended mitigation methods.

7.8 FIELD DATA CORRECTIONS AND NORMALIZATION

In-situ testing methods such as Standard Penetrometer Test (SPT), electronic Cone Penetrometer Test (CPT), electronic Piezocone Penetrometer Test with pore pressure readings (CPTu), and Flat Plate Dilatometer Test (DMT) may require corrections or adjustments prior using the results for soil property correlation or in design. These in-situ testing methods are described in Chapter 5. The SPT and CPT field data are the most commonly corrected or normalized to account for overburden pressure, energy, rod length, non-standard sampler configuration, borehole diameter, fines content, and the presence of thin very stiff layers. The data obtained from the DMT is corrected for the effects of the instrument operation on the results of the testing. All corrections for in-situ testing methods that are used in geotechnical design and analyses shall be documented in the geotechnical report. The following sections discuss corrections and adjustments in greater detail.

7.8.1 SPT Corrections

Many correlations exist that relate the corrected N-values to relative density (D_r), peak effective angle of internal friction (ϕ'), undrained shear strength (S_u), and other parameters; therefore it is incumbent upon the designer to understand the correlations being used and the requirements of the correlations for corrected N-values. Design methods are available for using N-values directly in the design of driven piles, embankments, spread footings, and drilled shafts. These corrections are especially important in liquefaction potential assessments (Chapter 13 – Geotechnical Seismic Hazards). Design calculations using SPT N-value correlations should be performed using corrected N-values, however, only the actual field SPT N_{meas}-values should be plotted on the soil logs and profiles depicting the results of SPT borings. Each of the corrections is discussed in greater detail in the following sections.

7.8.1.1 Energy Correction (C_E)

The type of hammer used to collect split-spoon samples must be noted on the boring logs. Typically correlations used between soil parameters and N-values are based on a hammer having an energy potential of 60 percent of the theoretical maximum. Typically a split-spoon sampler advanced with a manual safety hammer will have an approximate energy level of 60 percent (ER ≈ 60%). The energy ratio (ER) is the measured energy divided by the theoretical maximum (i.e. 140-pound hammer dropping 30 inches or 4,200 inch-pounds). The measured energy is determined as discussed in Chapter 5.

Split-spoon samples are also advanced with either an automatic hammer (ER ≈ 90%) or a donut hammer (ER ≈ 45%) **[<u>Reminder:</u> *The use of the donut hammer is not permitted*]**. The corrections for the donut hammer are provided for information only because some past projects were performed using the donut hammer. N-values obtained using either the automatic or the donut hammer will require correction prior to being used in engineering analysis. The energy correction factor (C_E) shall be determined using the following equation. Typical C_E values are provided in Table 7-2 for each hammer type. These correction factors should only be used when the actual hammer energy has not been previously measured.

$$C_E = \frac{ER}{60} \qquad \text{Equation 7-1}$$

Where ER is the measured energy expressed as an integer (i.e. 90 percent energy is ER = 90).

Table 7-2, Energy Ratio by Hammer Type (C_E)

Hammer Type	Energy Ratio (ER) %	C_E
Automatic	80	1.33
Safety	60	1.00
Donut	45	0.75

7.8.1.2 Overburden Correction (C_N)

N_{meas}-values will increase with depth due to increasing overburden pressure. The overburden correction is used to standardize all N-values to a reference overburden pressure. The reference overburden pressure is 1 ton per square foot (tsf) (1 atmosphere). The overburden correction factor (C_N) (Cetin et al., 2004) is provided below.

$$\mathbf{C_N} = \left(\frac{\mathbf{1}}{\sigma'_{\mathbf{V}}}\right)^{\mathbf{0.5}} \leq \mathbf{1.6} \qquad \text{Equation 7-2}$$

7.8.1.3 Rod Length Correction (C_R)

N_{meas}-values measured in the field should be corrected for the length of the rod used to obtain the sample. The original N_{60}-value measurements were obtained using long rods (i.e. rod length greater than 33 feet); therefore, a correction to obtain "equivalent" N_{60}-values for short rod length (i.e. rod length less than 33 feet) is required. Typically the rod length will be the depth of the sample (d) plus an assumed 7 feet of stick up above the ground surface. The rod length correction factor (C_R) equation is provided below with typical values presented in Table 7-3 (McGregor and Duncan, 1998).

$$C_R = e^{-e^{(-0.11d-0.77)}} \qquad \text{Equation 7-3}$$

Table 7-3, Rod Length Correction (C_R)

Rod Length (feet)	C_R
< 13	0.75
13 – 20	0.85
20.1 – 33	0.95
> 33	1.00

7.8.1.4 Sampler Configuration Correction (C_S)

The sampler configuration correction factor (C_S) (Cetin et al., 2004) is used to account for samplers designed to be used with liners, but the liners are omitted during sampling. If the sampler is not designed for liners or if the correct size liner is used no correction is required (i.e. C_S = 1.0). When liners are omitted there is an increase to the inside diameter of the sampler; therefore, the friction between the soil and the sampler is reduced. The sampler configuration correction factor is presented in Table 7-4.

Table 7-4, Sampler Configuration Correction (C_S)

Sampler Configuration	C_S
Standard Sampler not designed for liners	1.0
Standard Sampler design for and used with liners	1.0
Standard Sampler designed for liners and used without liners:	
$N_{meas} \leq 10$	1.1
$11 \leq N_{meas} \leq 29$	$1 + N_{meas}/100$
$30 \leq N_{meas}$	1.3

7.8.1.5 Borehole Diameter Correction (C_B)

The borehole diameter affects the N_{meas}-value if the borehole diameter is greater than 4.5 inches. Large diameter boreholes allow for stress relaxation of the soil materials. This stress relaxation can be significant in sands, but have a negligible effect in cohesive soils. Therefore, for cohesive soils use C_B equal to 1.0. Listed in Table 7-5 are the borehole diameter correction factors (C_B) (McGregor and Duncan, 1998).

Table 7-5, Borehole Diameter Correction (C_B)

Borehole Diameter (inches)	C_B
2-1/2 – 4-1/2	1.0
6	1.05
8	1.15

7.8.1.6 Fines Content Correction (C_F)

The N_{meas}-value may require correction for fines content (FC). This correction is applied during liquefaction analysis (see Chapter 13). It should be noted that a different fines correction is required for determination of seismic soil settlement (Chapter 13). The fines content correction (C_F) (Cetin et al., 2004) is determined by the following equation.

$$\mathbf{C_F} = (\mathbf{1} + \mathbf{0.004FC}) + \mathbf{0.05}\left(\frac{\mathbf{FC}}{\mathbf{N^*_{1,60}}}\right) \qquad \text{Equation 7-4}$$

Where FC is the percent fines content expressed as an integer (i.e. 15 percent fines is FC =15). This fines content correction factor is limited to fines contents between 5 percent and 35 percent ($5\% \leq FC \leq 35\%$). For fines content less than 5 percent use FC = 0 and for fines content greater than 35 percent use FC = 35. $N^*_{1,60}$ is defined in the following section.

7.8.1.7 Corrected N-values

As indicated previously the N-values measured in the field (N_{meas}) require corrections or adjustments prior to being used for the selection of design parameters or in direct design methods. The N-value requirements of the correlations or the direct design methods should be well understood and known to the engineer. Corrections typically applied to the N_{meas} -values are listed in the following equations.

$$\mathbf{N_{60} = N_{meas} \cdot C_E} \tag{Equation 7-5}$$

$$\mathbf{N_{1,60} = N_{60} \cdot C_N} \tag{Equation 7-6}$$

$$\mathbf{N^*_{60} = N_{meas} \cdot C_E \cdot C_R \cdot C_S \cdot C_B} \tag{Equation 7-7}$$

$$\mathbf{N^*_{1,60} = N^*_{60} \cdot C_N} \tag{Equation 7-8}$$

$$\mathbf{N^*_{1,60,CS} = N^*_{1,60} \cdot C_F} \tag{Equation 7-9}$$

7.8.2 <u>CPT Corrections</u>

The CPT tip resistance (q_c) and sleeve resistance (f_s) require corrections to account for the effect of overburden on the tip and sleeve resistance. The tip resistance may also be corrected to account for thin stiff layers located between softer soil layers. These corrections are discussed in the following sections.

7.8.2.1 Effective Overburden Normalization

The measured CPT tip resistance (q_c) and sleeve resistance (f_s) are influenced by the effective overburden stress. This effect is accounted for by normalizing the measured resistances to a standard overburden stress of 1 tsf (1 atm). The normalized CPT tip resistance ($q_{c,1}$) and sleeve resistance ($f_{s,1}$), are computed as indicated by the following equations.

$$\mathbf{q_{c,1} = C_q q_c} \tag{Equation 7-10}$$

$$\mathbf{f_{s,1} = C_q f_s} \tag{Equation 7-11}$$

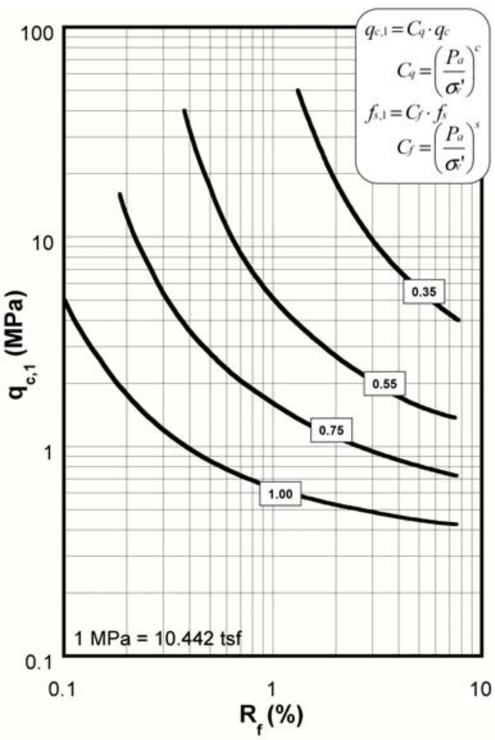
Where,

- q_c = Measured CPT tip resistance. Units of MPa (1 MPa ≅ 10.442 tsf)
- f_s = Measured CPT sleeve resistance. Units of MPa (1 MPa ≅ 10.442 tsf)
- C_q = Overburden normalization factor is the same for q_c and f_s as indicated in Equation 7-12.

$$\mathbf{C_q = \left(\frac{P_a}{\sigma'_v}\right)^c \leq 1.7} \tag{Equation 7-12}$$

Where,

- σ'_v = Effective overburden stress in units of tsf at the time that the CPT testing was performed. Future variations in water table or surcharges should not be included in the calculations.
- P_a = Atmospheric pressure, taken as 1 tsf (1 atm)
- c = Normalization exponent that can be determined from Figure 7-1.



$$c = f_1 \left(\frac{R_f}{f_3} \right)^{f_2} \qquad \text{Equation 7-13}$$

q_c = measured CPT tip resistance
R_f = friction ratio (f_s/q_c) in percent
f_s = measured CPT friction resistance

$$f_1 = 0.78(q_c)^{-0.33}$$

$$f_2 = \left(0.32(q_c)^{-0.35}\right) - 0.49$$

$$f_3 = abs\left[\log(10 + q_c)\right]^{1.21}$$

Iterative Calculation Method:

1. Select an initial estimate of the normalization exponent (c) using measured q_c using Figure 7-1(A) or Equation 7-13.
2. Normalize tip using Equation 7-11. (Note: R_f will not change when tip and friction sleeve are normalized equivalently)
3. Compute revised normalization exponent using Equation 7-13.
4. Iterate procedure until acceptable convergence.

(A) CPT Normalization Exponent Curves	**(B) Iterative Calculation Method of Normalization Exponent (*c*)**

Figure 7-1, Normalization of CPT Overburden Exponent (*c*) (Moss et al., 2006)

7.8.2.2 Thin Layer Correction

When the measured CPT tip resistance (q_c) is obtained in a thin layer of stiff soils that is embedded between softer surrounding soils, the measured tip resistance (q_c) will be reduced due to the effects of the underlying softer soils. This case commonly occurs in fluvial environments where granular soils are interbedded between layers of cohesive soils. Granular soils that are affected by this reduction in tip resistance (q_c) are typically sand layers that are less than 5 feet thick. The CPT tip resistance for this special case that is normalized and corrected for the thin layer ($q_{c,1,thin}$) and is computed as indicated in the following equation.

$$\mathbf{q_{c,1,thin}} = \mathbf{C_{thin}}(\mathbf{q_{c,1}}) \qquad \text{Equation 7-14}$$

Where,

$q_{c,1}$ = Measured CPT tip resistance. Units of MPa (1 MPa ≅ 10.442 tsf)

C_{thin} = Thin layer correction factor. The C_{thin} is determined from Figure 7-2 (See recommended bold red lines) based on the ratio of uncorrected q_c values for layers B and A (q_{cB}/q_{cA}) and the thickness of the thin layer (*h*). The value for C_{thin} should be limited to $C_{thin} \leq 1.8$ for thin layer thickness, $h < 5$ feet (1200 mm). A value of $C_{thin} = 1.0$ should be used for granular soil layers with a thickness, $h \geq 5$ feet (1200 mm). These corrections apply to a 10 cm² cone (diameter, d=35.7mm).

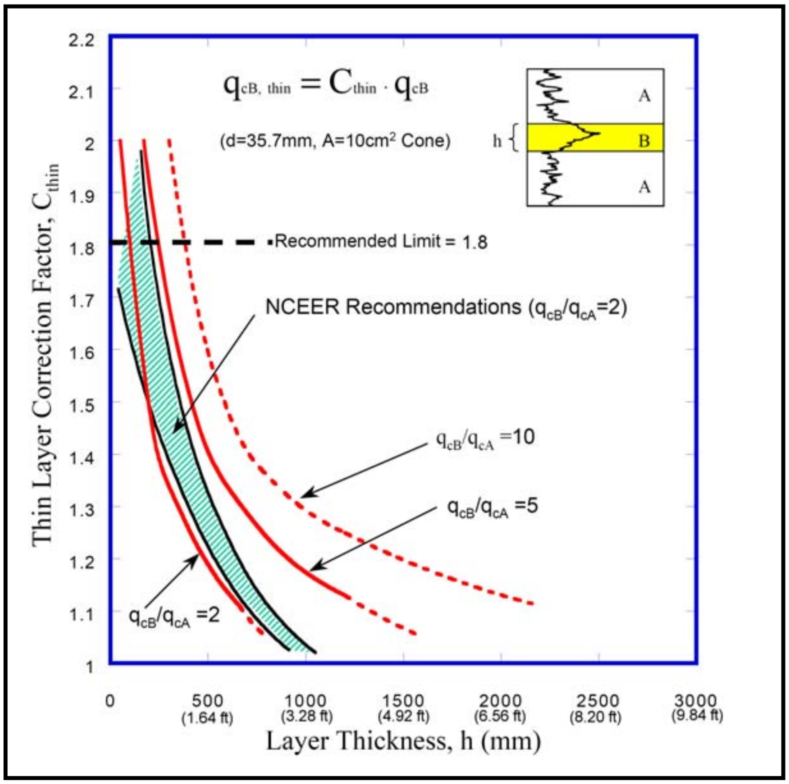


Figure 7-2, CPT Thin Layer Correction (C_{Thin})
(Moss et al., 2006)

In lieu of using Figure 7-2 the following equation may be used to compute the C_{thin}.

$$\mathbf{C_{thin} = A(304.878\,h)^B \leq 1.800 \quad \text{for } h < 5 \text{ feet and } \left(\frac{q_{cB}}{q_{cA}}\right) \leq 5} \qquad \text{Equation 7-15}$$

Where,

h = layer thickness in feet

$$A = 3.744\left(q_{cB}/q_{cA}\right)^{0.491}$$

$$B = -0.050\ln\left(q_{cB}/q_{cA}\right) - 0.204$$

$$\left(q_{cB}/q_{cA}\right) = \text{Stiffness Ratio}$$

7.8.2.3 Correlating CPT Tip Resistance To SPT N-Values

Since some design methodologies have only been developed for SPT blow counts, the CPT tip resistance is sometimes correlated to SPT blow counts. It is recommended that the normalized cone tip resistance, $q_{c,1}$, or the normalized cone tip resistance adjusted for the effects of "fines", $q_{c,1,mod}$, be normalized and corrected as indicated in Chapter 13 first and then correlated to normalized SPT values $N_{1,60}$ or $N_{1,60,cs}$. The following correlation by Jefferies and Davies (1993) should be used to correlate the CPT tip resistance to the SPT blow count.

$$N_{1,60} = \frac{q_{c,1}}{8.5\left(1 - \frac{I_c}{4.75}\right)}$$ Equation 7-16

$$N_{1,60,cs} = \frac{q_{c,1,mod}}{8.5\left(1 - \frac{I_c}{4.75}\right)}$$ Equation 7-17

Where,

- $q_{c,1}$ = Normalized CPT cone tip resistance Units of tsf. See Section 7.8.2.1.
- $q_{c,1,mod}$ = Normalized CPT cone tip resistance adjusted for "fines" Units of tsf. See Chapter 13.
- I_c = Soil behavior type.

The soil behavior type, I_c, is computed using normalized tip resistance (Q_T), normalized sleeve friction (F_R), and normalized pore pressure (B_q). The following equations should be used.

$$Q_T = \frac{q_{c,1} - \sigma_v}{\sigma_v'}$$ Equation 7-18

$$F_R = \frac{f_{s,1}}{\left(q_{c,1} - \sigma_v\right)} \times 100$$ Equation 7-19

$$B_q = \frac{\left(U_2 - U_0\right)}{\left(q_t - \sigma_v\right)}$$ Equation 7-20

Where,

- $q_{c,1}$ = Where q_c is the normalized CPT cone tip resistance, units of tsf.
- $f_{s,1}$ = Where f_s is the normalized CPT cone tip resistance, units of tsf.
- σ_v' = Effective overburden pressure, units of tsf
- σ_v = Total overburden pressure, units of tsf
- U_2 = Pore pressure measurement located on the tip shoulder, unit of tsf
- U_0 = Hydrostatic water pressure, units of tsf

The soil behavior type, I_c, is computed using the following equation.

$$\mathbf{I_c} = \sqrt{\left[\mathbf{3} - \mathbf{log}\left(\mathbf{Q_T}\left(\mathbf{1} - \mathbf{B_q}\right)\right)\right]^2 + \left[\mathbf{1.5} + \left(\mathbf{1.3}\,\mathbf{log}\left(\mathbf{F_R}\right)\right)\right]^2}$$ Equation 7-21

The soil behavior type, I_c, can be generally correlated to a soil classification as indicated in Table 7-6.

Table 7-6, Soil CPT Index (I_c) and Soil Classification

CPT Index (I_c)	Soil Classification
I_c <1.25	Gravelly Sands
$1.25 \le I_c < 1.90$	Sands – Clean Sand to Silty Sand
$1.90 \le I_c < 2.54$	Sand Mixtures – Silty Sand to Sandy Silt
$2.54 \le I_c < 2.82$	Silt Mixture – Clayey Silt to Silty Clay
$2.82 \le I_c < 3.22$	Clays

7.8.3 Dilatometer Corrections

The data A, B, and C pressure readings from the dilatometer require correction to account for the effects of the physical composition of the instrument (i.e. the stiffness of the membrane, new membranes are stiffer than used membranes). The horizontal stress index (K_D) shall be reported for all DMT results. The DMT corrections and computations for the horizontal stress index (K_D) shall be computed in accordance with FHWA-SA-91-044, *The Flat Dilatometer Test*, publication dated February 1992.

7.9 SOIL LOADING CONDITIONS AND SOIL SHEAR STRENGTH SELECTION

Geotechnical engineering as presented in this Manual has a statistical (LRFD) and performance-base design components that require selection of appropriate soil properties in order to design within an appropriate margin of safety consistent with Chapter 9 and also to predict as reasonable as possible the geotechnical performance required in Chapter 10. The selection of soil shear strengths by the geotechnical engineer requires that the designer have a good understanding of the loading conditions and soil behavior, high quality soil sampling and testing, and local geotechnical experience with the various geologic formations. This section provides guidance in the selection of shear strengths for cohesive soils (i.e. clays) and cohesionless soils (i.e. sands and nonplastic silts) for use in geotechnical design. The selection of shear strength parameters for rock is covered in the Section 7.14.

For an in-depth review of the topics addressed in this Section, see Sabatini et al. (2002) and Duncan and Wright (2005).

Geotechnical load resisting analyses that are typically performed in the design of transportation facilities are bearing capacity of a shallow foundation, axial (tension and compression) load carrying capacity of deep foundations (drilled shafts and piles), lateral carrying capacity of deep foundations, stability analyses of hillside slopes and constructed embankments, sliding resistance of earth retaining structures, and passive soil capacity resistance. Each of these analyses can have various loading conditions that are associated with the limit state (Strength, Service, Extreme Event) under evaluation.

Soil shear strength is not a unique property and must be determined based on the anticipated soil response for the loading condition being evaluated. This requires the following three-step evaluation process:

- **Evaluate the Soil Loading**: The soil loading should be investigated based on the soil loading rate, the direction of loading, and the boundary conditions for the limit state (Strength, Service, Extreme Event) being evaluated.

- **Evaluate Soil Response**: The soil response should be evaluated based on pore pressure build-up (Δu), the soil's state of stress, volumetric soil changes during shearing, and the anticipated magnitude of soil deformation or strain for the soil loading being applied.

- **Evaluate Appropriate Soil Strength Determination Method:** This consists of determining the most appropriate soil testing method that best models the loading condition and the soil response for determination of soil shear strength design parameters. Also included in this step is the review of the results for reasonableness based on available correlations and regional experience.

The three-step evaluation process is discussed in detail in the following Sections.

7.9.1 Soil Loading

The soil loading can be evaluated with respect to loading rate, direction of loading, and boundary conditions. The loading rate primarily affects the soils response with respect to pore water pressure build-up (Δu). When the loading rate either increases or decreases the pore water pressure ($\Delta u \neq 0$), the loading is referred to as short-term loading. Conversely, if the loading rate does not affect the pore water pressure ($\Delta u = 0$), the loading is referred to as a long-term loading.

Short-term loadings typically occur during construction such as when earth-moving equipment place large soil loads within a relatively short amount of time. The actual construction equipment (cranes, dump trucks, compaction equipment, etc.) should also be considered during the evaluation the construction loadings. Construction loadings are typically evaluated under the Strength limit state. Earthquakes or impacts (vessel or vehicle collisions) that can apply a significant amount of loading on the soil within a short amount of time are also referred to as short-term loadings. Because of the relative transient and infrequent nature of earthquake and impact loadings, geotechnical design for these types of loadings are performed under the Extreme Event limit states.

Long-term loadings are typically the result of static driving loads placed on the soils when performing limit state equilibrium analyses such as those that occur with embankments, retaining walls, or foundation that have been in place for a sufficient length of time that the pore water pressures have dissipated. These types of loadings are typically evaluated under the Strength and Service limit states.

The direction of loading is directly related to the critical failure surface and it's angle of incidence with respect to the soil element under evaluation. This becomes important when analyzing the soil shear strength with respect to a base of a retaining wall sliding over the foundation or during the analysis of soil stability where the failure surface intersects the soil at various angles within the soil mass. The shear strength is also affected by plane strain loading condition as is typically observed under structures such as continuous wall footings. Plane strain loading occurs when the strain in the direction of intermediate principal stress is zero.

Soil loading boundary conditions result from the soil-structure interaction between the loads imposed by the structure and the soil. The loadings and soil response are interdependent based on the stress-strain characteristics of the structure and the soil. Boundary conditions also include the frictional interface response between the structure and the soil. These boundary conditions can be very complex and affect the magnitude of the soil loadings, magnitude of the soil resistance, the distribution of the soil loading (rigid or flexible foundation), and the direction of the loading.

7.9.2 Soil Response

The soil response is influenced significantly by the soils pore water pressure response (Δu) resulting from the rate of loading as the soils attempt to reach a state of equilibrium. The undrained condition is a soil response that occurs when there is either an increase (+) in pore water pressure ($\Delta u > 0$) or a decrease (-) in pore water pressure ($\Delta u < 0$) within the soil during soil loading. The drained condition is a soil response that occurs when there is no change in pore water pressure ($\Delta u = 0$) as a result of the soil loading.

The pore water pressure response (Δu) that allows water to move in or out of the soil over time is dependent on the soil drainage characteristics and the drainage path. The time for drainage to occur can be estimated by using Terzaghi's theory of one-dimensional consolidation where the time required to reach 99% of the equilibrium volume change, t_{99}, is determined by the following equation.

$$t_{99} = 4\frac{D^2}{C_v}$$

Equation 7-22

Where,

- D = Longest distance that water must travel to flow out of the soil mass
- C_v = Coefficient of vertical consolidation (units length squared per unit of time)

Typical drainage times for various types of soil deposits based on Equation 7-22 are provided in Figure 7-3. It can readily be seen that cohesionless soils (sands) drain within minutes to hours while cohesive soils (clays) drain within months to years. Silty soils can drain within hours to days. Even though a soil formation may behave in an undrained condition at the beginning of the load application with excess pore water pressures ($\Delta u \neq 0$), with sufficient time to allow for pore pressure dissipation, the soils will reach a drained condition where static loads are in equilibrium and there is no excess pore water pressure ($\Delta u = 0$). Because soil layers may have different drainage characteristics and drainage paths within a soil profile, soil layers may be at various stages of drainage with some soil layers responding in an undrained condition while other layers respond in a drained condition.

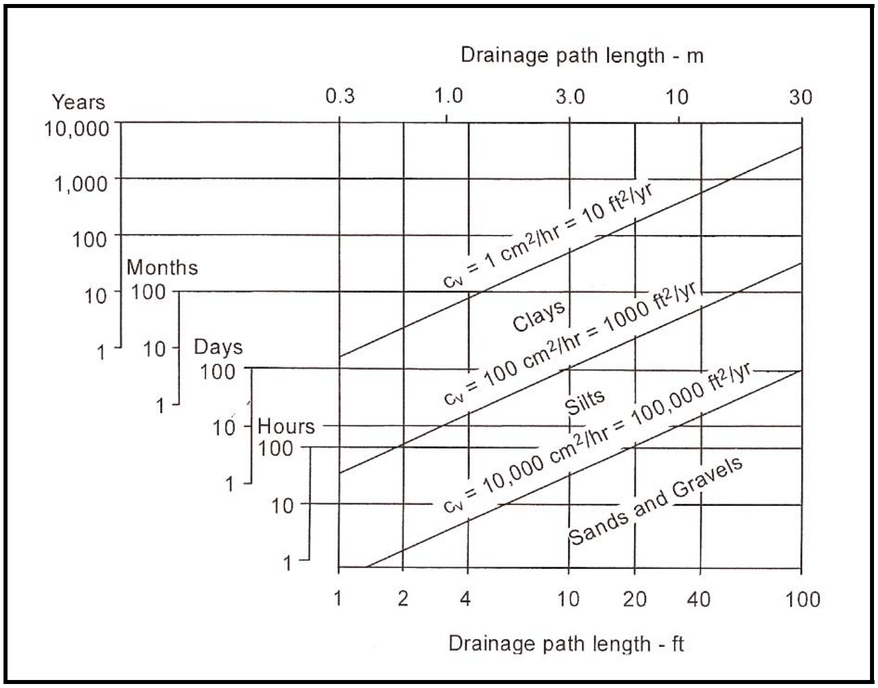


Figure 7-3, Drainage Time Required (Duncan and Wright, 2005)

There are various soil models that are used to characterize soil shear strength. The simplest and most commonly used soil shear strength model is the Mohr-Coulomb soil failure criteria. More sophisticated soil shear strength models such as critical state soil mechanics and numerical models (finite element constitutive soil models) exist and are to be used when simpler models such as the Mohr-Coulomb soil failure criteria cannot accurately predict the soil response.

When undrained conditions exist ($\Delta u \neq 0$), total stress parameters are used to evaluate soil shear strength. Total stress is characterized by using total shear strength parameters (c, ϕ) and total stress, σ_{vo}, (total unit weights). The basic Mohr-Coulomb soil failure criteria for total stress shear strength (τ), also referred to as the undrained shear strength (S_u), is shown in the following equation.

$$\tau = \mathbf{c} + \sigma_{\mathbf{vo}} \ \mathbf{tan}\,\phi \qquad \text{Equation 7-23}$$

Where,

c	=	Total soil cohesion.
σ_{vo}	=	Total vertical overburden pressure. Total unit weights (γ_T) are used.
ϕ	=	Total internal soil friction angle. The total internal soil friction angle for cohesive soils is typically assumed to equal zero (ϕ = 0). Total internal soil friction angle (ϕ) for a cohesionless soil is typically less than the effective internal soil friction angle (ϕ').

When drained conditions exist ($\Delta u = 0$), effective stress parameters are used to evaluate soil shear strength. Effective stress is characterized by using effective shear strength parameters (c', ϕ') and effective stress, σ'_{vo}, (effective unit weights). The basic Mohr-Coulomb soil failure criteria for effective stress shear strength (τ') is shown in the following equation.

$$\tau' = c' + \sigma'_{vo} \tan\phi' \qquad \text{Equation 7-24}$$

Where,

- c' = Effective soil cohesion. The effective cohesion for cohesive and cohesionless soils is typically assumed to equal zero ($c' = 0$).
- σ'_{vo} = Effective vertical overburden pressure. Effective unit weights ($\gamma' = \gamma_T - \gamma_w$) are used.
- ϕ' = Effective internal soil friction angle. The effective internal soil friction angle (ϕ') for a cohesionless soil is typically greater than the total internal soil friction angle (ϕ).

Another factor that affects soil response of cohesive soils is the in-situ stress state. The stress state is defined by either total (σ_{vo}) or effective (σ'_{vo}) vertical stress, total (σ_{ho}) or effective (σ'_{ho}) horizontal stress, and the effective preconsolidation stress (σ'_p or p'_c). The effective preconsolidation stress is the largest state of stress that the soil has experienced. The state of stress is often quantified by the overconsolidation ratio (*OCR*) as indicated by the following equation.

$$\mathbf{OCR} = \frac{\boldsymbol{\sigma}'_{p}}{\boldsymbol{\sigma}'_{vo}} \qquad \text{Equation 7-25}$$

Cohesive soils are often defined by the following in-situ state of stress:

- **Normally Consolidated (NC; *OCR* = 1):** If the effective overburden stress (σ'_{vo}) is approximately equal to the effective preconsolidation stress (σ'_p).

- **Overconsolidated (OC; *OCR* > 1):** If the effective overburden stress (σ'_{vo}) is less than the effective preconsolidation stress (σ'_p)

- **Underconsolidated (UC; *OCR* < 1):** If the effective overburden stress (σ'_{vo}) is greater than the effective preconsolidation stress (σ'_p)

Volumetric change (δ_v) during shearing can significantly affect the shear strength behavior of the soils. When the soil response is a decrease ($-\delta_v$) in volume during soil shearing the soils are termed to have ***contractive*** behavior. Loose sands and soft clays typically have contractive behavior. When the soil response is an increase ($+\delta_v$) in volume during soil shearing these soils are termed to have ***dilative*** behavior. Overconsolidated clays and medium-dense sands typically have dilative behavior. Soils that do not exhibit volumetric change during shearing (δ_v = 0) are termed to have ***steady state*** behavior.

For typical cohesive or cohesionless soils it has been observed that the soil shear stress (τ) varies as the soil strains or deforms during soil shearing. Selection of the appropriate soil shear strength to be used in design must be compatible with the deformation or strain that the soil will exhibit under the loading. This is best illustrated in Figure 7-4 where the drained stress-strain behavior of two stress-strain curves, each curve representing a different effective consolidation stress (σ'_{v1} and σ'_{v2}), are shown. On the left of Figure 7-4 is a shear stress vs. shear strain plot (τ-γ_s plot). Because there is a well-defined peak shear stress (τ_{max}) in the plots this would be indicative of dilative soil behavior of either dense sand or overconsolidated clay. The maximum shear stress (τ_{max}) is termed the ***peak shear strength*** ($\tau_{Peak} = \tau_{max}$). In overconsolidated clay soils, as the maximum shear stress (τ_{max}) is exceeded, post-peak strain softening occurs until a ***fully-softened strength*** (τ_{NC}) is reached. The fully-softened strength is a post-peak strain softening strength that is considered to be the shear strength that is equivalent to peak shear strength of the same soil in normally consolidated (NC) stress state ($\tau_{Peak} \approx \tau_{NC}$). For very large shearing strains in soils (cohesive or cohesionless) the shear stress value is reduced further to a residual shear strength (τ_r). The Mohr-Coulomb effective shear strength envelopes for peak shear strength ($\tau_{Peak} = \tau_{max}$), fully-softened shear strength ($\tau_{Peak} \approx \tau_{NC}$), and residual shear strength (τ_r) are illustrated on the right side of Figure 7-4.

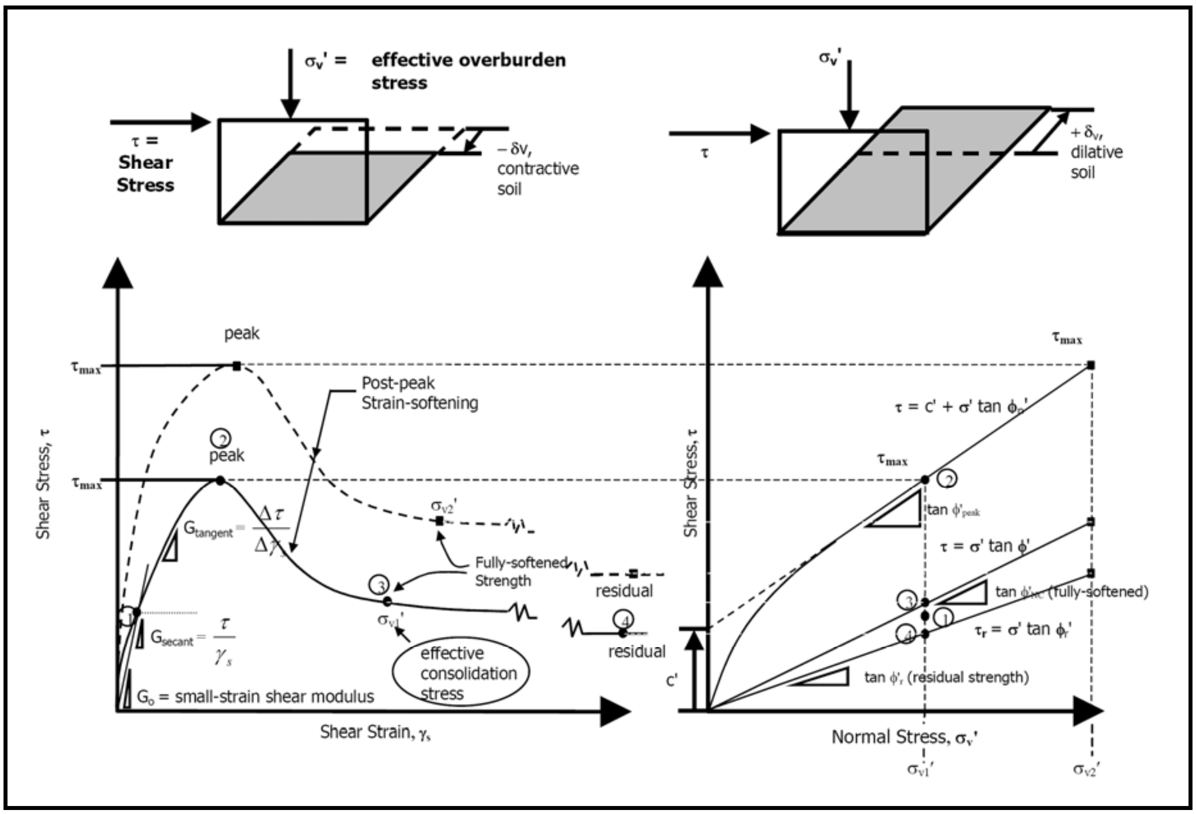


Figure 7-4, Drained Stress-Strain Behavior
(Sabatini et al., 2002)

The soil behavior of typical cohesionless soils can be further illustrated by comparing the stress-strain behavior of granular soils with various densities as shown in Figure 7-5. Medium and dense sands typically reach a peak shear strength ($\tau_{Peak} = \tau_{max}$) value and then decrease to a residual shear strength value at large displacements. The volume of medium and dense sands initially decreases (contractive behavior) and then increases as the soil grains dilate (dilative behavior) with shear displacement until it reaches a point of almost constant volume (steady state behavior). The shear stress in loose sands increases with shear displacement to a maximum value and then remains constant. The volume of loose sands gradually decreases (contractive behavior) until it reaches a point of almost constant volume (steady state behavior).

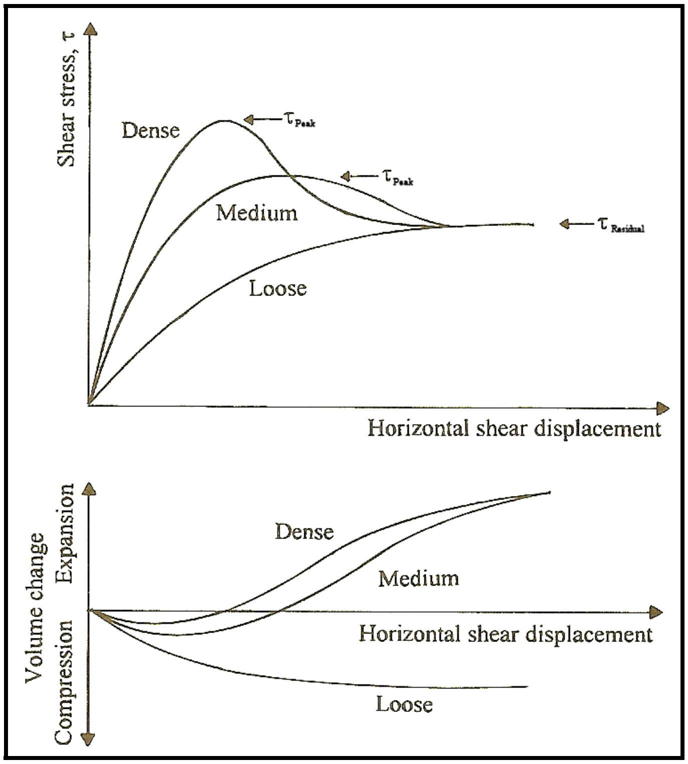


Figure 7-5, Shear Strength Sands (Direct Shear-Test) (Das, 1997)

The soil behavior of typical cohesive soils can be further illustrated by comparing the stress-strain behavior of normally consolidated clays (*OCR* = 1) with the stress-strain behavior of overconsolidated clays (*OCR* > 1) for consolidated drained and undrained Triaxial tests in Figures, 7-6 and 7-7, respectively. The stress-strain behavior for overconsolidated clays (*OCR* > 1) indicates that they are subject to strain softening, similar to medium-dense sands shown in Figure 7-5, and that normally consolidated clays (*OCR* = 1) increases in strength, similar to loose sands also shown in Figure 7-5. Overconsolidated (drained or undrained) clays typically reach peak shear strength ($\tau_{Peak} = \tau_{max}$) and then decrease to a fully-softened strength that is approximately equal to the peak shear strength of a normally consolidated clay ($\tau_{Peak} \approx \tau_{NC}$). The volume change of overconsolidated clays in a drained test is very similar to the volume change in medium-dense sand; the volume initially decreases (contractive behavior) and then increases (dilative behavior). The pore pressures in an undrained test of overconsolidated clays initially increase slightly and then become negative as the soil begins to expand or dilate. The shear stress (drained or undrained test) of a normally consolidated (OCR = 1) clay increases with shear displacement to a maximum value ($\tau_{Peak} = \tau_{NC}$). The volume of normally consolidated clays in a drained test gradually decreases (contractive behavior) as it reaches a point of almost constant volume (steady state behavior). The pore pressure in an undrained test of normally consolidated clay increases until failure and remains positive for the entire test.

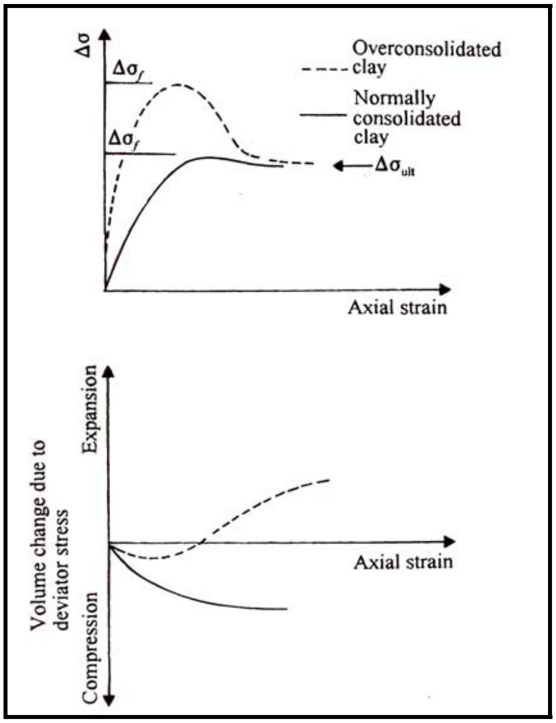


Figure 7-6, Shear Strength of Clay Consolidated Drained Triaxial
(Das, 1997)

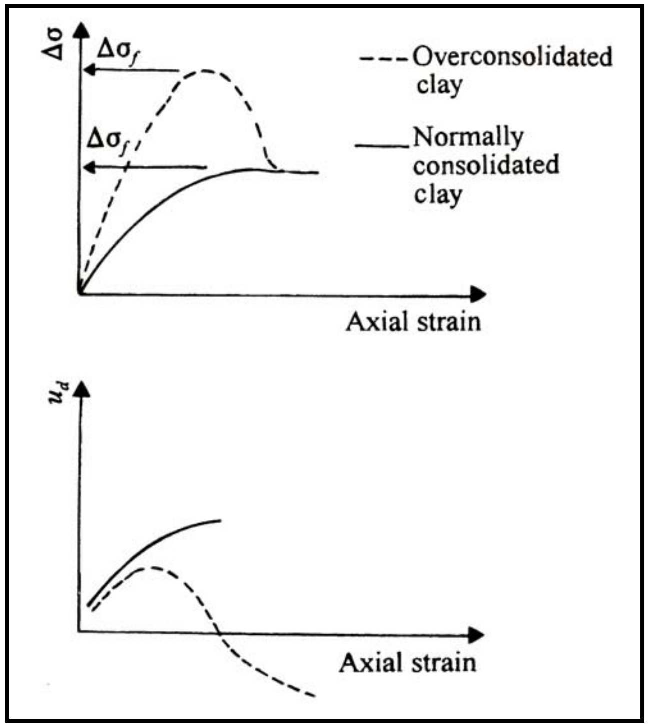


Figure 7-7, Shear Strength of Clay Consolidated Undrained Triaxial
(Das, 1997)

Selection of soil shear strengths should be made based on laboratory testing and soil strain level anticipated from analyses. Table 7-7 provides a summary of published stress-strain behavior from Holtz and Kovacs (1981), Terzaghi, Peck, and Mesri (1996), and Duncan and Wright (2005) for various soils types. This table is provided for "general" guidance in the selection of shear strengths and soil strain level anticipated from equilibrium analyses.

Table 7-7, Soil Shear Strength Selection Based on Strain Level

Cohesive Soils (Undrained)	Strain Level [1]		
	±2% Strains	10–15% Strains	Large Strains >15%
Clay (*OCR*=1)	$\tau_{Peak} = \tau_{NC}$	$\tau_{Peak} = \tau_{NC}$	$\tau_{Peak} = \tau_{NC}$
Clay (*OCR*>1)	τ_{Peak}	$\approx \tau_{NC}$	τ_r
Cohesionless Soils (Drained)	**Strain Level [1]**		
	±5% Strains	**15–20% Strains**	**Large Strains >20%**
Med. To Dense Sand	τ_{Peak}	τ_r	τ_r
Non-Liquefying Loose Sands	τ_{Peak}	τ_{Peak}	τ_r

Shear Strength Nomenclature:
τ_{Peak} = Peak Soil Shear Strength τ_{NC} = Normally Consolidated Soil Shear Strength
τ_r = Residual Soil Shear Strength
[1] Strain levels indicated are generalizations and are dependent on the stress-strain characteristics of the soil and should be verified by laboratory testing.

7.9.3 Soil Strength Testing

Once the soil loading and soil response has been evaluated the next step is to select the method of evaluating the soil shear strength. The shear strength can be evaluated by one of the following methods:

- Soil shear strength determined by geotechnical laboratory testing
- Soil shear strength correlations with in-situ field testing results
- Soil shear strength correlations based on index parameters

The laboratory testing should be selected based on shear strength testing method and the testing parameters best suited to model the loading condition and the soil response. Shear strength laboratory testing methods are described in Chapter 5. A summary of the design parameters that should be used in selection of the appropriate testing method and procedure is provided below:

- **Total or Effective Stress:** Selection of soil shear strength parameters based on total or effective stress state (drained or undrained). Guidance for typical geotechnical analyses for each limit state (Strength, Service, and Extreme Event) being analyzed is provided for bridge foundations in Table 7-8 and for earth retaining structures and embankments in Table 7-9. Total and effective shear strength determination guidelines for laboratory and in-situ testing are provided in Sections 7.10 and 7.11, respectively.

- **Soils Shear Strength:** Soil shear strength parameters (τ_{Peak} or τ_r) selection should be based on strain level anticipated from equilibrium analyses. See Table 7-7 for guidance. Seismic soil shear strengths used to design for the Extreme Event I limit state are discussed in Chapter 12.

- **Loading Direction:** The shearing direction should be compatible with how the soil is being loaded or unloaded and the angle of incidence with respect to soil normal stress. Figure 7-8 illustrates test methods that would be appropriate for shear modes for embankment instability shear surface. Figure 7-9 provides undrained strength (UU Triaxial) of typical clays and shales as a function of stress orientation.

Table 7-8, Bridge Foundation Soil Parameters

Limit State		Strength		Service	Extreme Event			
Load Combinations		Strength I, II, III, IV, V		Service I	Extreme Event I			
Seismic Event		N/A			FEE & SEE			
Loading Condition		Static			During Earthquake Shaking		Post-Earthquake	
Soil Shear Strength Stress State		Total	Effective	Effective	Total [1]	Drained	Total [1]	Drained
Shallow Foundation Design	Soil Bearing Resistance	√	√	---	√	√	√	---
	Sliding Frictional Resistance	√	√	---	√	√	√	---
	Sliding Passive Resistance	√	√	---	√	√	√	---
	Structural Capacity	√	√	---	√	√	√	---
	Lateral Displacement	---	---	√	√	√	√	---
	Vertical Settlement	---	---	∇	∇	∇	∇	∇
	Overall Stability	---	---	√	√	√	√	---
Deep Foundation Design	Axial Capacity	√	•	---	---	√	√	---
	Structural Capacity	√	√	---	---	√	√	---
	Lateral Displacements	---	---	√	√	√	√	---
	Vertical Settlement	---	---	∇	∇	∇	∇	∇

[1] Residual soil shear strengths of liquefied soils must include effects of strain softening due to liquefaction.

Soil Stress State Legend:

√ Indicates that soil stress state indicated requires analysis

--- Indicates that soil stress state does not require analysis

• Indicates that soil stress state may need to be evaluated depending on method of analysis

∇ Indicates that soil stress state transitions from undrained to drained (i.e. consolidation)

Table 7-9, Earth Retaining Structures & Embankment Soil Parameters

	Limit State	Strength		Service		Extreme Event			
	Load Combinations	Strength I, II, III, IV, V		Service I		Extreme Event I			
	Seismic Event	N/A				FEE & SEE			
	Loading Condition	Static				During Earthquake Shaking		Post-Earthquake	
	Soil Shear Strength Stress State	Total	Effective	Total	Effective	Total [1]	Effective	Total [1]	Effective
Earth Retaining Structure Design	Soil Bearing Resistance	√	√	---	---	√	√	---	√
Earth Retaining Structure Design	Sliding Frictional Resistance	√	√	---	---	√	√	---	√
Earth Retaining Structure Design	Sliding Passive Resistance	√	√	---	---	√	√	---	√
Earth Retaining Structure Design	Structural Capacity	√	√	---	---	√	√	---	√
Earth Retaining Structure Design	Lateral Load Analysis (Lateral Displacements)	---	---	√	√	√	√	---	√
Earth Retaining Structure Design	Settlement	---	---	∇	∇	∇	∇	∇	∇
Earth Retaining Structure Design	Global Stability	---	---	√	√	√	√	---	√
Embankment Design	Soil Bearing Resistance	√	√	---	---	√	√	---	√
Embankment Design	Lateral Spread	√	√	---	---	√	√	---	√
Embankment Design	Lateral Squeeze	√	√	---	---	√	√	---	√
Embankment Design	Lateral Displacements	---	---	√	√	√	√	---	√
Embankment Design	Vertical Settlement	---	---	∇	∇	∇	∇	∇	∇
Embankment Design	Global Stability	---	---	√	√	√	√	---	√

[1] Residual soil shear strengths of liquefied soils must include effects of strain softening due to liquefaction

<u>Soil Stress State Legend:</u>

√ Indicates that soil stress state indicated requires analysis

--- Indicates that soil stress state does not require analysis

• Indicates that soil stress state may need to be evaluated depending on method of analysis

∇ Indicates that soil stress state transitions from undrained to drained (i.e. consolidation)

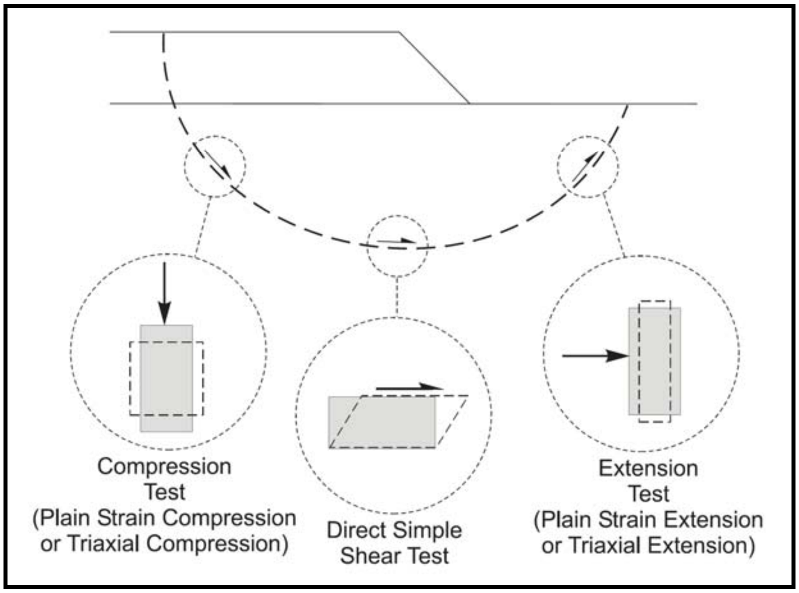


Figure 7-8, Shear Modes for Embankment Stability Shear Failure Surface (Sabatini, 2005)

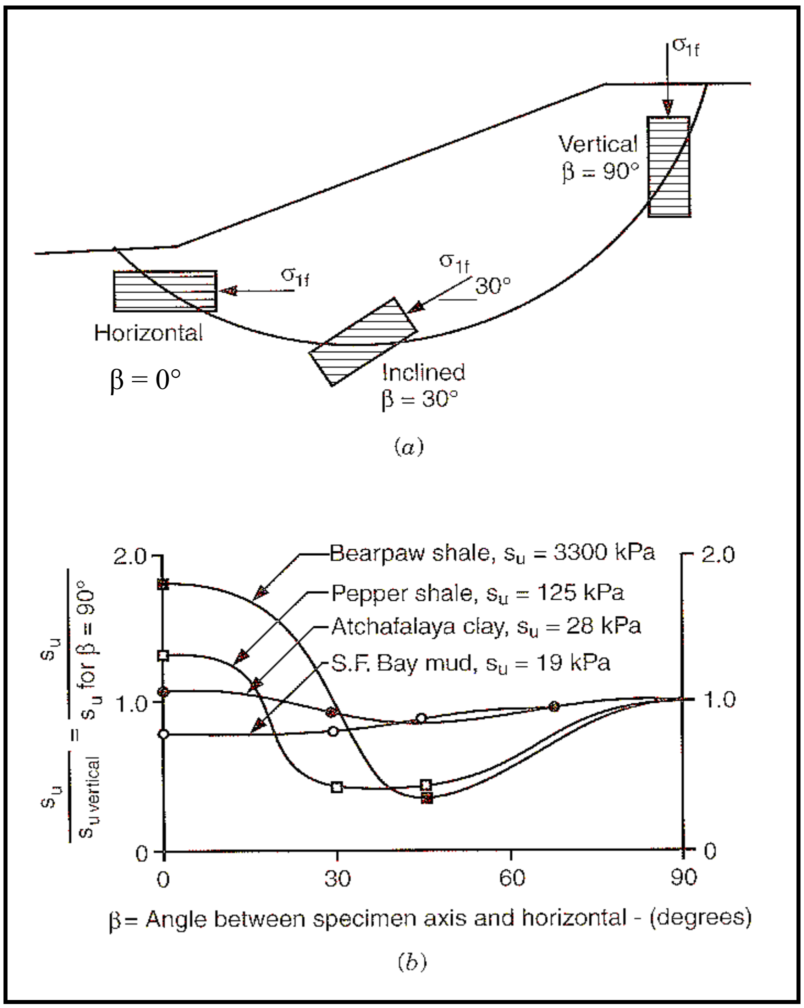


Figure 7-9, τ of Clays and Shales as Function of Failure Orientation (modified from Duncan and Wright, 2005)

The undrained and drained shear strengths of soils can be obtained from laboratory testing. The laboratory testing procedures are described in Chapter 5. A summary of laboratory testing methods suitable for determining the undrained and drained shear strengths of cohesive and cohesionless soils is provided in Table 7-10.

Table 7-10, Laboratory Testing Soil Shear Strength Determination

Laboratory Testing Method	Undrained Shear Strength				Drained Shear Strength			
	Cohesive		Cohesionless		Cohesive		Cohesionless	
	τ_{Peak}	τ_r	τ_{Peak}	τ_r	τ'_{Peak}	τ'_r	τ'_{Peak}	τ'_r
Unconfined Compression (UC) Test	√	√	---	---	---	---	---	---
Unconsolidated Undrained (UU) Test	√	√	---	---	---	---	---	---
Consolidated Drained (CD) Test	---	---	---	---	---	---	√	√
Consolidated Undrained (CU) Test with Pore Pressure Measurements	√	√	√	√	√	√	√	√
Direct Shear (DS) Test	---	---	---	---	---	---	√	√

√ - Indicates laboratory method provides indicated shear strength
--- - N/A
Definitions:
τ_{Peak} = Peak Undrained Shear Strength
τ_r = Residual Undrained Shear Strength
τ'_{Peak} = Peak Drained Shear Strength
τ'_r = Residual Drained Shear Strength

In-situ testing methods (Section 5.3) such as Standard Penetrometer Test (SPT), electronic Cone Penetrometer Test (CPT), electronic Piezocone Penetrometer Test (CPTu – CPT with pore pressure readings), Flat Plate Dilatometer Test (DMT), and Vane Shear Test (VST), can be used to evaluate soil shear strength parameters by the use of empirical/semi-empirical correlations. Even though the torvane (TV) or the pocket penetrometer (PP) are soil field testing methods, their use is restricted to only qualitative evaluation of relative shear strength during field visual classification of soil stratification. The major drawback to the use of in-situ field testing methods to obtain soil shear strength parameters is that the empirical/semi-empirical correlations are based on a limited soil database that is typically material or soil formation specific and therefore the reliability of these correlations must be verified for each project site until sufficient substantiated regional experience is available. Poor correlation between in-situ testing results and soil shear strength parameters may also be due to the poor repeatability of the in-situ testing methods. The electronic Cone Penetrometer Test (CPT) has been shown to be more repeatable while the Standard Penetration Test (SPT) has been shown to be highly variable. Another source of variability is the sensitivity of the test method to different soil types with different soil consistency (very soft to hard cohesive soils) or density (very loose to very dense cohesionless soils). In-situ penetration testing values correspond to the peak of the stress-strain shear strength curve as indicated in Figure 7-10. Since deformations induced from penetration tests are close to the initial stress state, correlations have been developed for the soil modulus.

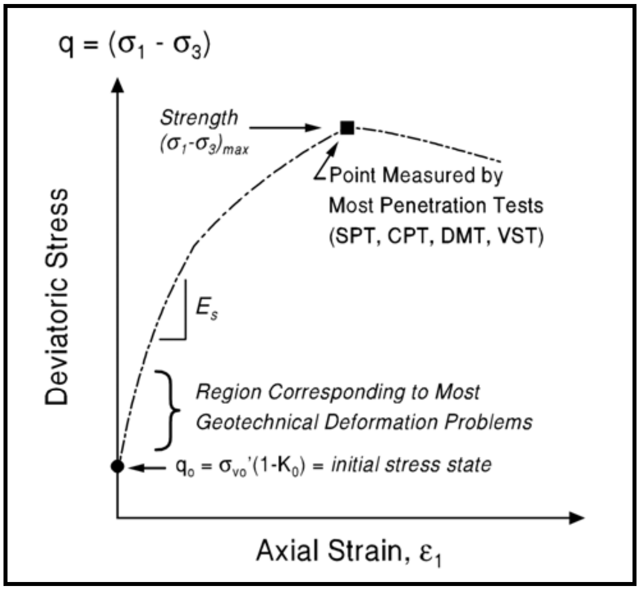


Figure 7-10, Shear Strength Measured by In-Situ Testing (Sabatini, 2005)

A summary of in-situ testing methods suitable for determining the undrained and drained shear strengths of cohesive and cohesionless soils is provided in Table 7-11. The suitability of in-situ testing methods to provide soil shear strength parameters is provided in Table 7-12.

Table 7-11, In-Situ Testing - Soil Shear Strength Determination

In-Situ Testing Method	Undrained Shear Strength				Drained Shear Strength			
	Cohesive		Cohesionless		Cohesive		Cohesionless	
	τ_{Peak}	τ_r	τ_{Peak}	τ_r	τ'_{Peak}	τ'_r	τ'_{Peak}	τ'_r
Standard Penetrometer Test (SPT)	√	---	---	---	---	---	√	---
Cone Penetrometer Test (CPT) or Piezocone with pore pressure measurements (CPTu)	√	√	---	---	---	---	√	---
Flat Plate Dilatometer Test (DMT)	√	---	---	---	---	---	√	---
Vane Shear Test (VST)	√	√	---	---	---	---	---	---

√ - Indicates in-situ method provides indicated shear strength
--- - N/A

Definitions:

τ_{Peak} = Peak Undrained Shear Strength
τ_r = Residual Undrained Shear Strength
τ'_{Peak} = Peak Drained Shear Strength
τ'_r = Residual Drained Shear Strength

Table 7-12, Soil Suitability of In-Situ Testing Methods (Modified from Canadian Geotechnical Manual (1982) and Holtz and Kovacs (1981))

In-Situ Test Method	Suitable Soils [1]	Unsuitable Soils	Correlated Properties	Remarks
Standard Penetrometer Test (SPT)	Sand, Clay, Residual Soils	Gravel	Sand and residual soil effective peak internal friction angle, clay undrained peak shear strength, soil modulus.	SPT repeatability is highly variable. Disturbed samples. Very variable S_u correlations are available for clays.
Cone Penetrometer Test (CPT) or Piezocone with pore pressure measurements (CPTu)	Sand, Silt, Clay, Residual Soil	Gravel	Sand, silt, and residual soil effective peak internal friction angle, clay and residual soil undrained peak shear strength, soil modulus.	Continuous evaluation of soil properties. CPT is very repeatable. No samples recovered.
Flat Plate Dilatometer Test (DMT)	Sand, Clay, and Residual Soil	Gravel	Sand, silt, and residual soil effective peak internal friction angle, clay and undrained peak shear strength, overconsolidation ratio, at-rest pressure coefficient, soil modulus.	Unreliable results may occur with very dense sand, cemented sand, and gravel. No samples recovered.
Vane Shear Test (VST)	Clay	Sand, Residual Soil, and Gravel	Clay undrained peak shear strength.	May overestimate shear strength. Very soft clays need to be corrected. Unreliable results may occur with fissured clays, varved clays, highly plastic clays, sand, residual soil, and gravel. VST repeatability may be variable with rate of rotation. No samples recovered.

[1] The suitability of testing Piedmont residual soils should be based on Mayne et al. (2000). Residual soils frequently have a dual USCS description of SM-ML and behave as both cohesive soils and cohesionless soils because the Piedmont residuum soil is close to the opening size of the U.S. No. 200 Sieve (0.075 mm).

Shear strength of cohesive and cohesionless soils can also be estimated based on effective overburden stress (σ'_{vo}), effective preconsolidation stress (σ'_p or p'_c), the overconsolidation ratio (OCR), and index properties such as grain size distribution (Fines Content – FC), moisture content (*w*), and Atterberg Limits (LL, PI). Index properties are described in Chapter 5. Unless indicated otherwise, these correlations are used only for preliminary analyses or for evaluating accuracy of laboratory or in-situ shear strength results.

7.10 TOTAL STRESS

Total stress is the force per unit area carried by both the soil grains and the water located in the pores between the soil grains. The total stress state uses undrained soil shear strengths ($\Delta u \neq 0$) and is typically used to resist short-term loadings (i.e. construction loading, earthquake loadings, etc.). The Mohr-Coulomb undrained shear strength equation ($\tau = S_u$) is defined as follows:

$$\tau = \mathbf{c} + \sigma_{\mathbf{v}} \tan \phi$$ Equation 7-26

The deviator compression stress at failure ($\Delta\sigma_f$) for unconfined compression tests ($\sigma_3 = 0$) on clays is equal to the unconfined compression strength ($\sigma_1 = q_u = c$). The deviator compression stress at failure ($\Delta\sigma_f$) for undrained triaxial testing (unconsolidated or consolidated) is equal to the total major principal stress (σ_1) minus the total minor principal stress (σ_3) (see Figure 7-11).

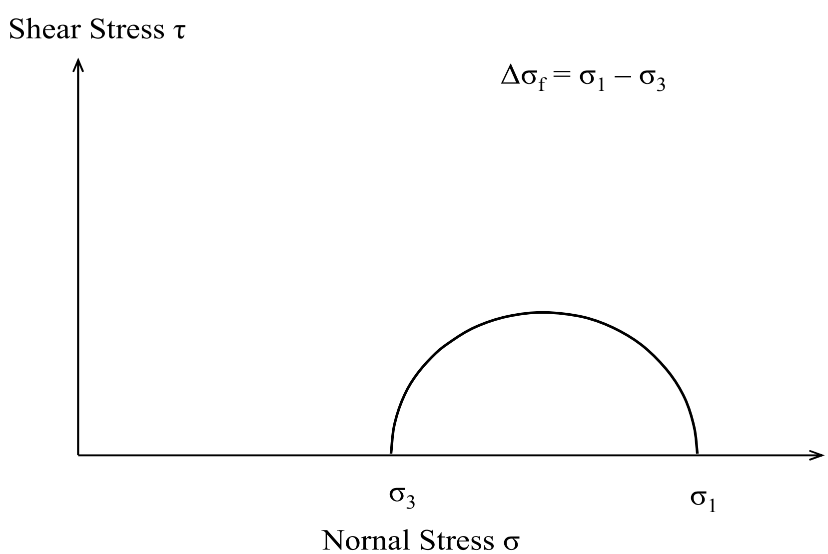


Figure 7-11, Total Principal Stresses

7.10.1 Cohesionless Soils

Undrained shear strengths of cohesionless soils (i.e. sand, low plasticity silts and residual soils) should be used when the rate of loading is so fast that the soil does not have sufficient time to drain such as in the case of rapid draw-down, cyclic loadings, earthquake loadings, and impact loadings. Geotechnical analyses for these types of loadings should use undrained shear strength parameters based on total stress analyses. The peak undrained shear strength in saturated cohesionless soils (τ_{Peak}) is also referred to in literature as the yield shear strength (τ_{yield}). The undrained peak shear strength (τ_{yield}) and the undrained residual shear strength (τ_r) of saturated cohesionless soils can be measured by conducting a consolidated undrained (CU) triaxial compression tests.

The peak undrained shear strength of cohesionless soils may also be determined by correlations developed for in-situ testing such as Standard Penetrometer Test (SPT) or Cone Penetrometer Test (CPT) as indicated in Chapter 5. As stated previously, in Section 7.9.3, the biggest drawback to the use of in-situ field testing methods to obtain undrained shear strengths of cohesionless soils is that the empirical correlations are based on a soil database that is material or soil formation specific and therefore the reliability of these correlations must be verified for each project site by substantiated regional experience or by conducting laboratory testing and calibrating the in-situ testing results.

Correlations have been proposed by Olson and Stark (2003) that relate yield strength ratio ($\tau_{yield}/\sigma'_{vo}$) to normalized SPT blowcount ($N^*_{1,60}$) and normalized CPT tip resistance ($q_{c,1}$). Where τ_{yield}, is the undrained peak shear strength of saturated cohesionless soils and σ'_{vo} is effective overburden pressure. Olson and Stark (2003) used case histories of static loading-induced failures and deformation-induced flow failures to assess the yield strength ratio ($\tau_{yield}/\sigma'_{vo}$).

The Olson and Stark (2003) relationship between yield shear strength ratio ($\tau_{yield}/\sigma'_{vo}$) and the normalized SPT blowcount ($N^*_{1,60}$) is provided in Figure 7-12. The average trend line for Figure 7-12 can be computed using the following equation.

$$\left(\frac{\tau_{\mathbf{yield}}}{\sigma'_{\mathbf{vo}}}\right) = \mathbf{0.205} + \mathbf{0.0075}\left(\mathbf{N}^*_{\mathbf{1,60}}\right) \pm \mathbf{0.04} \qquad \text{Equation 7-27}$$

Where,

$N^*_{1,60} \leq 12$ blow per foot

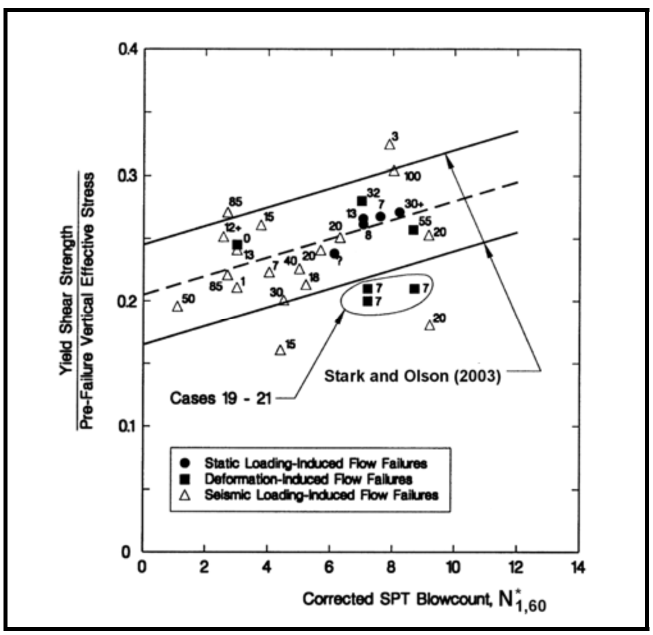


Figure 7-12, Yield Shear Strength Ratio - SPT Blowcount Relationship (Olson, 2001, Olson and Stark, 2003)

The Olson and Stark (2003) relationship between yield shear strength ratio ($\tau_{yield}/\sigma'_{vo}$) and the normalized CPT tip resistance ($q_{c,1}$) is provided Figure 7-13. The average trend line for Figure 7-13 can be computed using the following equation.

$$\left(\frac{\tau_{\text{yield}}}{\sigma'_{\text{vo}}}\right) = \mathbf{0.205} + \mathbf{0.0143}\ (\mathbf{q}_{\mathbf{c,1}}) \pm \mathbf{0.04} \qquad \text{Equation 7-28}$$

Where,

$q_{c,1} \leq 6.5$ MPa ≈ 68 tons per square foot (tsf)

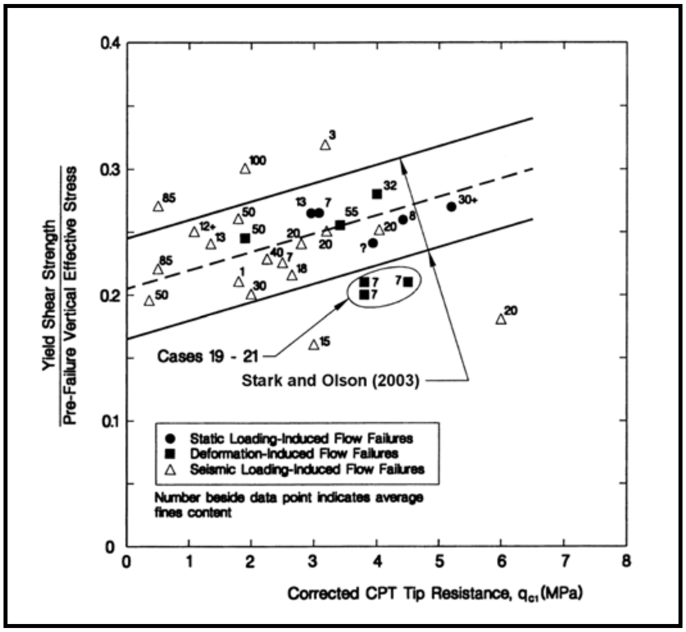


Figure 7-13, Yield Shear Strength Ratio - CPT Tip Resistance Relationship (Olson, 2001, Olson and Stark, 2003)

Undrained residual shear strength ratio of liquefied soils (τ_{rl}/σ'_{vo}) as proposed by Olson and Stark (2002, 2003) are presented in Chapter 12.

7.10.2 Cohesive Soils

The undrained shear strength (τ) of cohesive soils (i.e. clay, highly plastic silts and residual soils) can be determined using unconfined compression (UC) tests, unconsolidated undrained (UU) triaxial tests, or consolidated undrained (CU) triaxial tests of undisturbed samples. Typically the total internal friction angle is negligible and assumed equal to zero ($\phi = 0$) and the Mohr-Coulomb shear strength equation for the undrained shear strength (τ) of cohesive soils can be expressed as indicated by the following equation.

$$\tau = \mathbf{c} = \frac{\Delta\sigma_{\mathbf{f}}}{\mathbf{2}} \qquad \text{Equation 7-29}$$

The undrained shear strength of cohesive soils may also be determined by in-situ testing such as Standard Penetrometer Test (SPT), Cone Penetrometer Test (CPT), Flat Plate Dilatometer Test (DMT), or Vane Shear Test (VST) as described in Chapter 5. As stated previously, in Section 7.9.3, the biggest drawback to the use of in-situ field testing methods to obtain undrained shear strengths of cohesive soils is that the empirical correlations are based on a soil database that is material or soil formation specific and therefore the reliability of these correlations must be verified for each project site by substantiated regional experience or by conducting laboratory testing and calibrating the in-situ testing results.

The Standard Penetration Test (SPT) can provide highly variable results in cohesive soils as indicated in Table 7-10. However, the following correlations may be used if laboratory undrained shear strengths are correlated to the corrected N_{60} value obtained from the Standard Penetration Test. Peak undrained shear strength (τ), in units of ksf, for cohesive soils (McGregor and Duncan, 1986) can be computed for low plasticity clays using Equation 7-30 and medium to high plasticity clays using Equation 7-31. Plasticity is defined in Chapter 6.

$$\tau = \mathbf{c} = \mathbf{0.075\,N_{60}}$$ Equation 7-30

$$\tau = \mathbf{c} = \mathbf{0.15\,N_{60}}$$ Equation 7-31

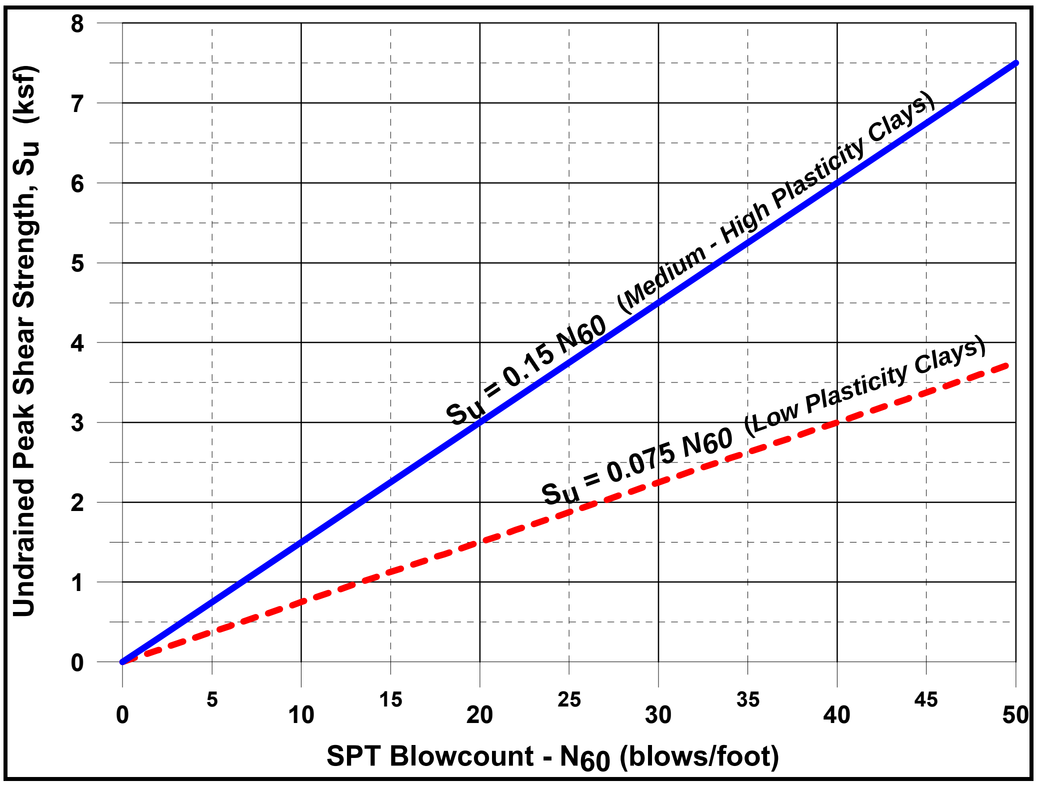


Figure 7-14, Undrained Shear Strength – SPT Relationship (McGregor and Duncan, 1986)

The peak undrained shear strength (τ) of cohesive soils can also be obtained from the Cone Penetrometer Test (CPT) (Sabatini, 2005) as indicated by the following equation.

$$\tau = \mathbf{c} = \frac{\mathbf{q_c} - \sigma_{\mathbf{vo}}}{\mathbf{N_k^*}} \qquad \text{Equation 7-32}$$

Where,

q_c = CPT tip resistance (measured, uncorrected)
σ_{vo} = total overburden pressure at test depth
N^*_k = cone factor.

The cone factor has been found to be approximately equal to 14 ± 5. Because of the large variation in N^*_k, CPT testing results shall be correlated with soil borings and laboratory testing to back-calculate the cone factor for the specific soil types under evaluation.

The Flat Plate Dilatometer Test (DMT) results should be corrected and correlated to undrained shear based on the FHWA Publication FHWA-SA-91-044, *The Flat Dilatometer Test*.

The peak undrained shear strength (τ) of cohesive soils can also be obtained from the Vane Shear Test (VST) (Aas et al., 1986) can be used as indicated by the following equation.

$$\tau = \mu \mathbf{S_{vane}} \qquad \text{Equation 7-33}$$

Where,

μ = Vane correction factor (see Figure 7-15)
S_{vane} = VST field measured undrained shear strength. The S_{vane} interpretation results should be based on ASTM STP1014 (1988).

The VST field measured undrained shear strength, S_{vane}, should be computed based on the following equation.

$$\boldsymbol{S_{vane}} = \frac{6\boldsymbol{T}}{\left(7\pi \boldsymbol{D}^3\right)} \quad \boldsymbol{for}\ \frac{\boldsymbol{H}}{\boldsymbol{D}} = 2 \qquad \text{Equation 7-34}$$

Where,

T = VST torque resistance
D = Diameter of field vane
H = Height of field vane

The vane correction factor (μ) is determined from the Aas et al. (1986) relationship shown in Figure 7-15. The vane correction factor (μ) is computed by entering the top chart with PI and (S_{vane}/σ'_{vo}) to establish whether the clay is within the normally consolidated (NC) range between the limits “young” and “aged”, or overconsolidated (OC). The lower chart is used by entering the (S_{vane}/σ'_{vo}) and selecting the vane correction factor (μ) for the appropriate NC or OC curves. A maximum vane correction factor (μ) of 1.0 is recommended by Aas, et. al (1986).

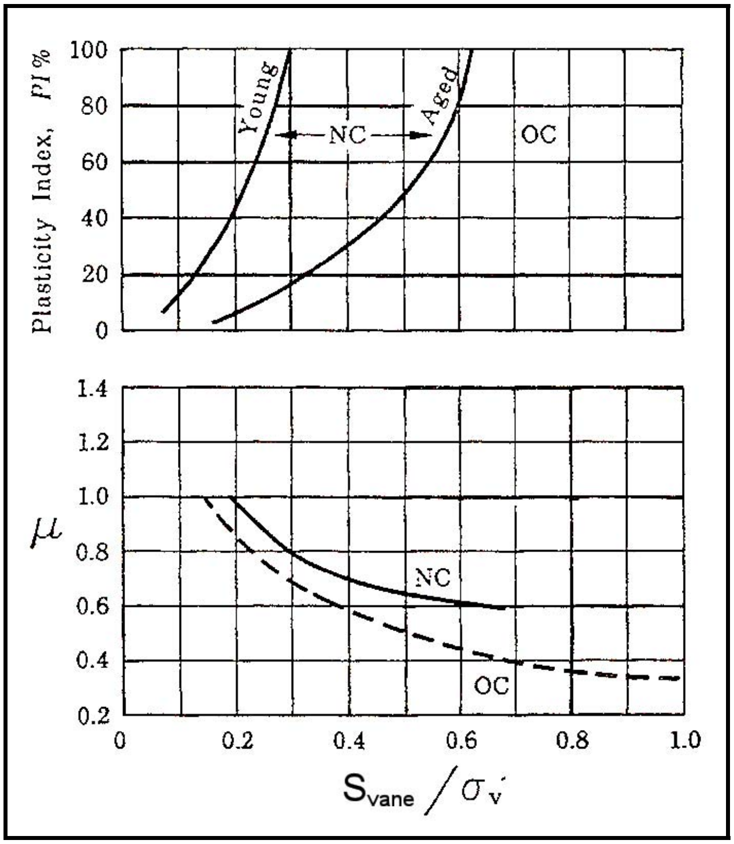


Figure 7-15, Vane Shear Correction Factor (Aas, et. al., 1986)

Empirical correlations based on SHANSHEP laboratory testing results can be used for preliminary designs and to evaluate the peak undrained shear strength (S_u) obtained from laboratory testing or in-situ testing. This method is only applicable to clays without sensitive structure where undrained shear strength increases proportionally with the effective overburden pressure (σ'_{vo}). The SHANSHEP laboratory test results of Ladd et al. (1977) revealed trends in undrained shear strength ratio (S_u / σ'_v) as a function of overconsolidation ratio as indicated in Figure 7-16.

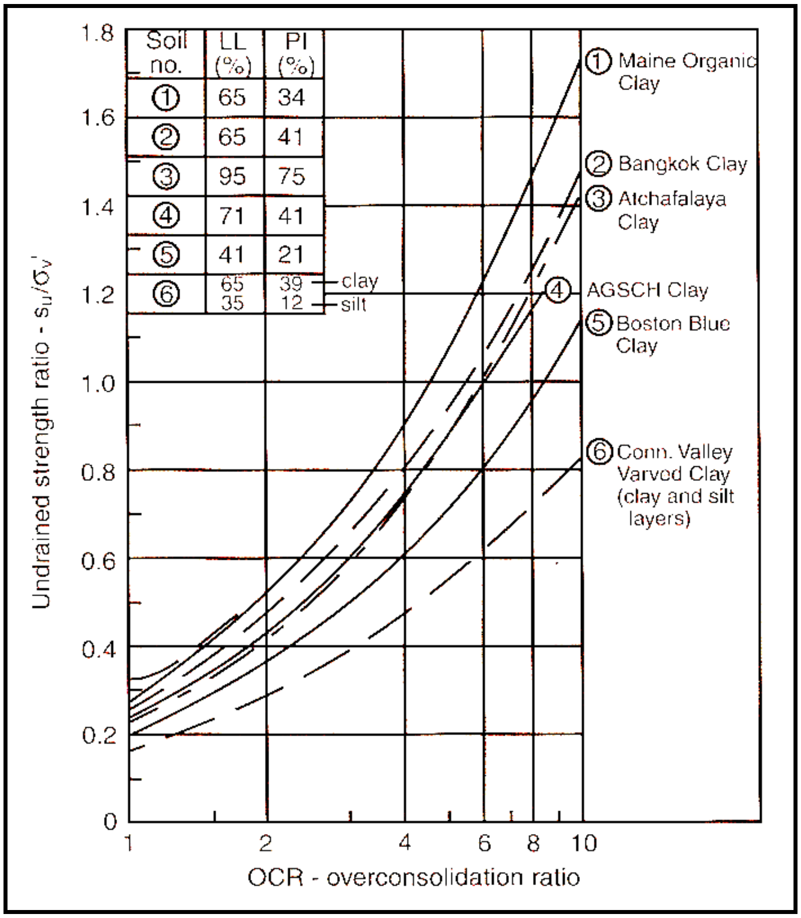


Figure 7-16, Undrained Shear Strength Ratio and OCR Relationship (Ladd et al., 1977)

The average peak undrained shear strengths (τ) shown in Figure 7-16 can be approximated by an empirical formula developed by Jamiolkowski et al. (1985) as indicated by the following equation.

$$\tau = \left(0.23(OCR)^{0.8}\right)\sigma'_{vo} \qquad \text{Equation 7-35}$$

Where,

τ = undrained shear strength (tsf)

OCR = overconsolidation ratio

σ'_{vo} = effective overburden pressure at test depth (tsf)

The undrained shear strength (τ) can be compared to the remolded shear strength (τ_R) (residual undrained shear strength, τ_r) to determine the sensitivity (S_t) of cohesive soils. Sensitivity is the measure of the breakdown and loss of interparticle attractive forces and bonds within cohesive soils. Typically in dispersed cohesive soils the loss is relatively small, but in highly flocculated structures the loss in strength can be large. Sensitivity is determined using the following equation.

$$\mathbf{S_t} = \frac{\tau}{\tau_\mathbf{R}}$$ Equation 7-36

The description of sensitivity is defined in the following table.

Table 7-13, Sensitivity of Cohesive Soils (Modified from Spangler and Handy, 1982)

Sensitivity	Descriptive Term
< 1	Insensitive
1 - 2	Slightly Sensitive
3 - 4	Medium Sensitive
5 - 8	Sensitive
9 - 16	Very Sensitive
17 - 32	Slightly Quick
33 - 64	Medium Quick
>64	Quick

The remolded shear strength of cohesive soils (τ_R) can be determined from remolded triaxial specimens or from in-situ testing methods (electro-piezocone or field vane). Triaxial specimens should have the same moisture content as the undisturbed sample as well as the same degree of saturation and confining pressure. Further sensitivity can be related to the liquidity index using the following figure.

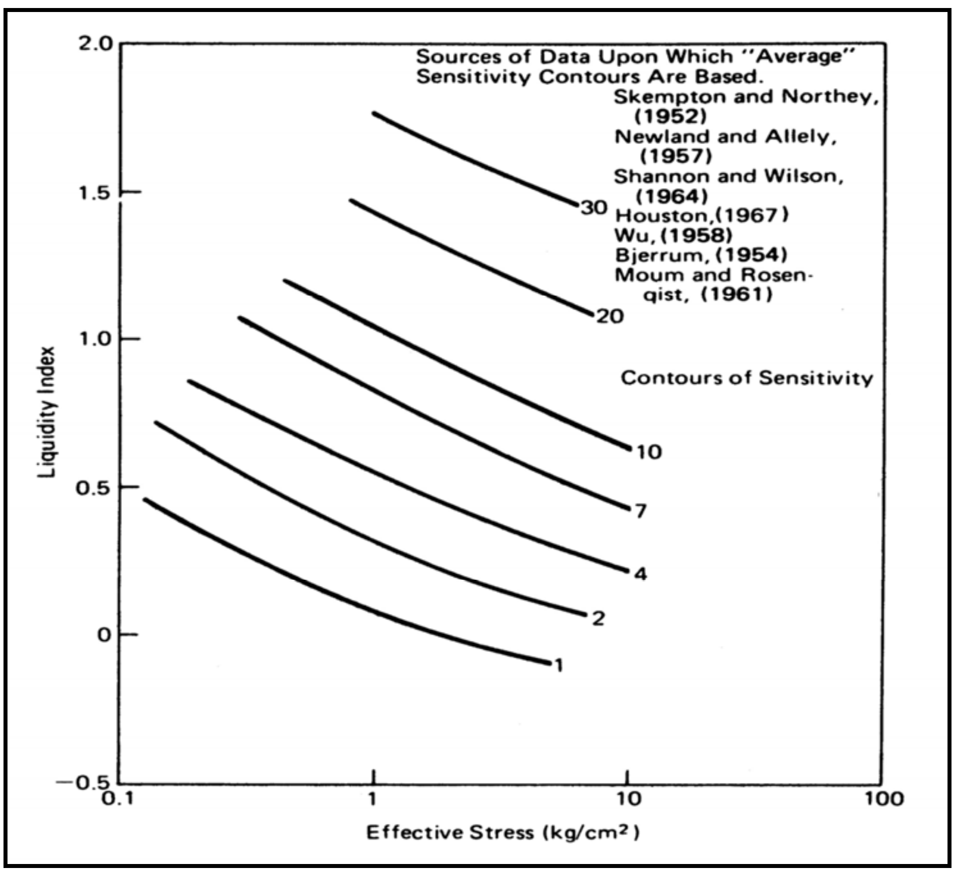


Figure 7-17, Sensitivity based on Liquidity Index and σ'_{vo} (Mitchell, 1993)

The Liquidity Index (LI) can also be related to remolded shear strength (τ_R = c_{ur} = S_{ur}) as indicated in the following.

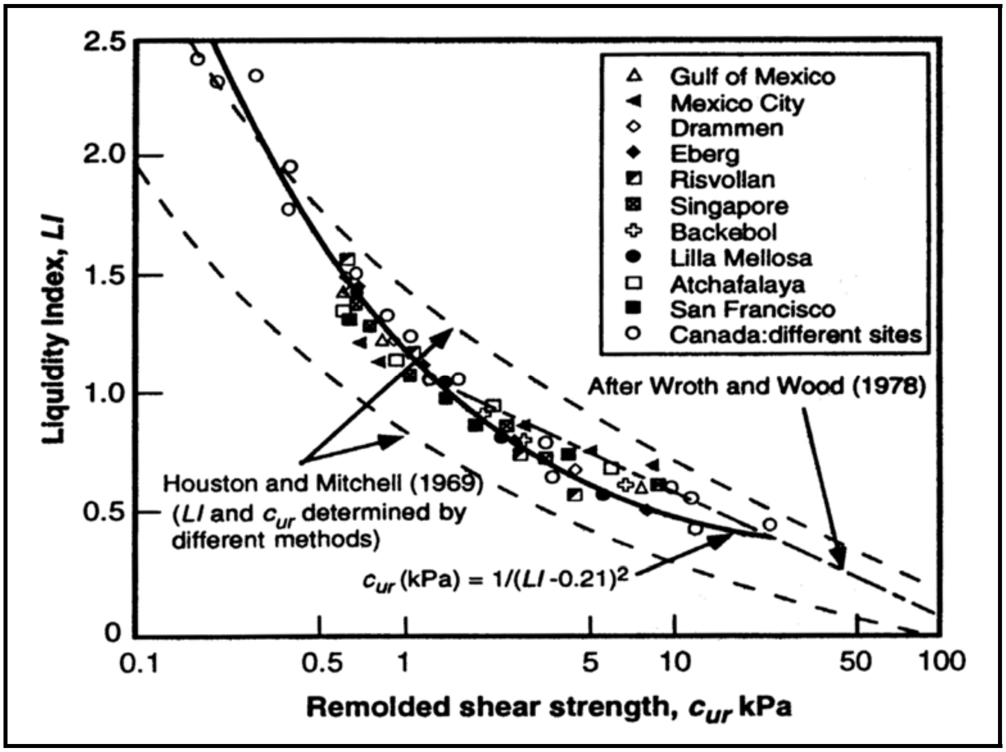


Figure 7-18, Remolded Shear Strength vs Liquidity Index (Mitchell, 1993)

Where,

1 kPa = 0.0209 ksf

The Liquidity Index (LI) is the relationship between natural moisture content, Plastic Limit (PL), and the Liquid Limit (LL). The LI is a measure of the relative softness of a cohesive soil as indicated by the closeness of the natural moisture content to the liquid limit. The LI can be determined by the following equation.

$$\mathbf{LI} = \frac{(\mathbf{w} - \mathbf{PL})}{(\mathbf{LL} - \mathbf{PL})} \qquad \text{Equation 7-37}$$

Where,

w = natural moisture content
LL = Liquid Limit
PL = Plastic Limit

The undrained residual shear strength of cohesive soils (S_t < 2) can be estimated for preliminary design and to evaluate the undrained residual shear strength (τ_r = S_{ur}) obtained from laboratory testing or in-situ testing. The undrained residual shear strength (τ_r = S_{ur}) can be estimated by reducing peak undrained shear strength (τ) by a residual shear strength loss factor (λ) as indicated in the following equation.

$$\tau_r = \lambda \tau$$ Equation 7-38

The residual shear strength loss factor (λ) typically ranges from 0.50 to 0.67 depending on the type of clay soil. The residual shear strength loss factors (λ) recommended in Table 7-14 are based on the results of a pile soil set-up factor study prepared by Rauche et al. (1996)

Table 7-14, Residual Shear Strength Loss Factor (λ)

Soil Type		Residual Shear Strength Loss Factor (λ)
USCS	**Description**	
Low Plasticity Clay	CL-ML	0.57
Medium to High Plasticity Clay	CL & CH	0.50

7.10.3 ϕ-c Soils

The undrained shear strength of soils that have both ϕ and c components should be determined in the laboratory using the appropriate testing methods. However, if the samples for this type of testing have not been obtained (e.g. during the preliminary exploration), then the soil should be treated as if the soil were either completely cohesive or cohesionless. For soils that are difficult to determine the approximate classification, the undrained shear strength parameters for both cohesive and cohesionless soils should be determined and the more conservative design should be used.

7.10.4 Maximum Allowable Total Soil Shear Strengths

SCDOT has established maximum allowable peak (c, ϕ) and residual (c_r, ϕ_r) undrained soil shear strength design parameters shown in Table 7-15, for use in design. These soil shear strength design parameters may not be exceeded without laboratory testing and the express written permission of the PCS/GDS.

Table 7-15, Maximum Allowable Total Soil Shear Strengths

Soil Type		Peak		Residual	
USCS	**Description**	***c*** **(psf)**	**ϕ (degrees)**	**c_r (psf)**	**ϕ_r (degrees)**
GW, GP, GM, GC	Stone and Gravel	0	34	0	18
SW	Coarse Grained Sand	0	17	0	7
SM, SP	Fine Grained Sand	0	17	0	7
SP	Uniform Rounded Sand	0	15	0	6
ML, MH, SC	Silt, Clayey Sand, Clayey Silt	1,500	15	1,200	6
SM-ML	Residual Soils	900	14	700	6
CL-ML	NC Clay (Low Plasticity)	1,500	0	900	0
CL, CH	NC Clay (Med-High Plasticity)	2,500	0	1250	0
CL-ML	OC Clay (Low Plasticity)	2,500	0	1400	0
CL, CH	OC Clay (Med-High Plasticity)	4,000	0	2000	0

7.11 EFFECTIVE STRESS

Effective stress is the force per unit area carried by the soil grains. The effective stress state uses drained soil shear strengths ($\Delta u = 0$). The Mohr-Coulomb drained shear strength equation is defined as follows.

$$\tau' = \mathbf{c}' + \sigma'_{\mathbf{v}} \ \mathbf{tan}\,\phi' \qquad \text{Equation 7-39}$$

The deviator compression stress at failure ($\Delta\sigma_f$) for undrained triaxial testing (consolidated) is equal to the total or effective major principal stress (σ_1) minus the total or effective minor principal stress (σ_3). The effective major and minor principal stresses are the total major and minor principal stresses minus the pore pressure at failure (u_f) (see Figure 7-19).

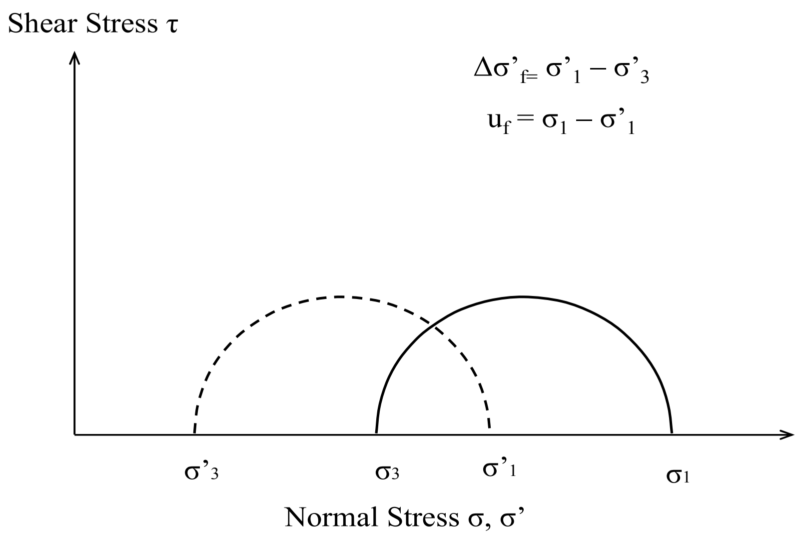


Figure 7-19, Effective Principal Stresses

7.11.1 Cohesionless Soils

Drained shear strengths of cohesionless soils (i.e. sand, low plasticity silts, and residual soils) should be used when there is relatively no change in pore water pressure ($\Delta u \approx 0$) as a result of soil loading. Cohesionless soils that are subjected to construction loads and static driving loads typically use peak or residual drained shear strengths due to the relatively rapid (minutes to hours) drainage characteristics of granular soils as indicated in Section 7.9.2. The peak or residual drained soil shear strength parameters can be obtained from consolidated drained (CD) triaxial tests, consolidated undrained (CU) triaxial tests with pore pressure measurements, or direct shear (DS) tests. Typically the effective cohesion (c') is negligible and assumed to be equal to zero ($c' = 0$) and the Mohr-Coulomb shear strength criteria for drained shear strength of cohesionless soils can then be expressed as indicated in the following equation.

$$\tau' = \sigma'_v \ \tan\phi' \qquad \text{Equation 7-40}$$

The peak drained shear strength of cohesionless soils may also be determined by in-situ testing methods such as the Standard Penetrometer Test (SPT), Cone Penetrometer Test (CPT), or Flat Plate Dilatometer Test (DMT). As stated previously, in Section 7.9.3, the biggest drawback to the use of in-situ field testing methods to obtain drained shear strengths of cohesionless soils is that the empirical correlations are based on a soil database that is material or soil formation specific and therefore the reliability of these correlations must be verified for each project site by either using substantiated regional experience or conducting laboratory testing and calibrating the in-situ testing results.

The effective peak friction angle, ϕ', of cohesionless soils can be obtained from Standard Penetrometer Test (SPT). Most SPT correlations were developed for clean sands and their use for micaceous sands/silts, silty soils, and gravelly soils may be may be unreliable as indicated below:

- SPT blow counts in micaceous sands or silts may be significantly reduced producing very conservative correlations.
- SPT blow counts in silty soils may produce highly variable results and may require verification by laboratory triaxial testing depending on a sensitivity analysis of the impact of the variability of results on the analyses and consequently the impact on the project.
- SPT blow counts in gravelly soils may overestimate the penetration resistance. Conservative selection of shear strength parameter or substantiated local experience should be used in lieu of laboratory testing.

The effective peak friction angle, ϕ', of cohesionless soils can be estimated using the relationship of Hatanaka and Uchida (1996) for corrected N-values ($N^*_{1,60}$) as indicated by Figure 7-20.

$$\phi' = \left[15.4 N^*_{1,60}\right]^{0.5} + 20^{\circ}$$ Equation 7-41

Where,
4 blows per foot $\leq N^*_{1,60} \leq$ 50 blows per foot

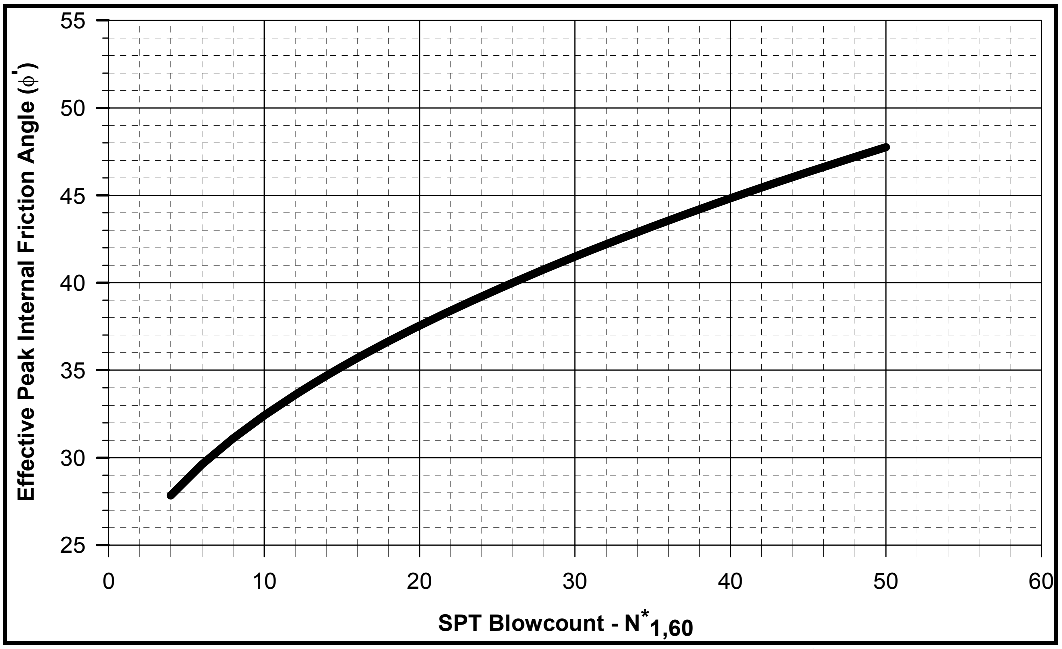


Figure 7-20, Effective Peak Friction Angle and SPT ($N^*_{1,60}$) Relationship (Based on Hatanaka and Uchida, 1996)

The effective friction angle, ϕ', of cohesionless soils can also be estimated by Cone Penetrometer Test (CPT) based on Robertson and Campanella (1983). This method requires the estimation of the effective overburden pressure (σ'_{vo}) and the cone tip resistance (q_c) measured, uncorrected using the relationship in Figure 7-21. This relationship may be approximated by the following equation.

$$\phi' = \tan^{-1}\left[0.1 + 0.38\log\left(\frac{q_c}{\sigma'_{vo}}\right)\right] \qquad \text{Equation 7-42}$$

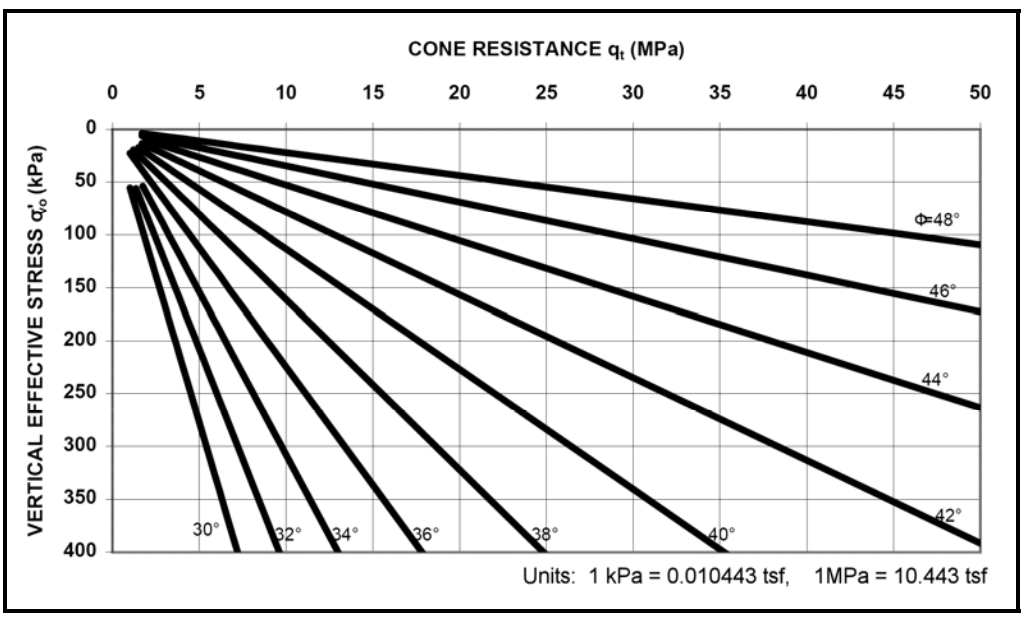


Figure 7-21, Effective Peak Friction Angle and CPT (q_c) Relationship (Robertson and Campanella, 1983)

The effective friction angle, ϕ', of cohesionless soils can also be estimated by Flat Plate Dilatometer Test (DMT) using the Robertson and Campanella (1991) relationship shown in Figure 7-22. This method requires the determination of the horizontal stress index (K_D) by the procedures outlined in FHWA-SA-91-044, *The Flat Plate Dilatometer*. The Robertson and Campanella (1991) relationship may be approximated by the following equation.

$$\phi' = 28^{\circ} + 14.6\log(K_D) - 2.1\log^2(K_D)$$ Equation 7-43

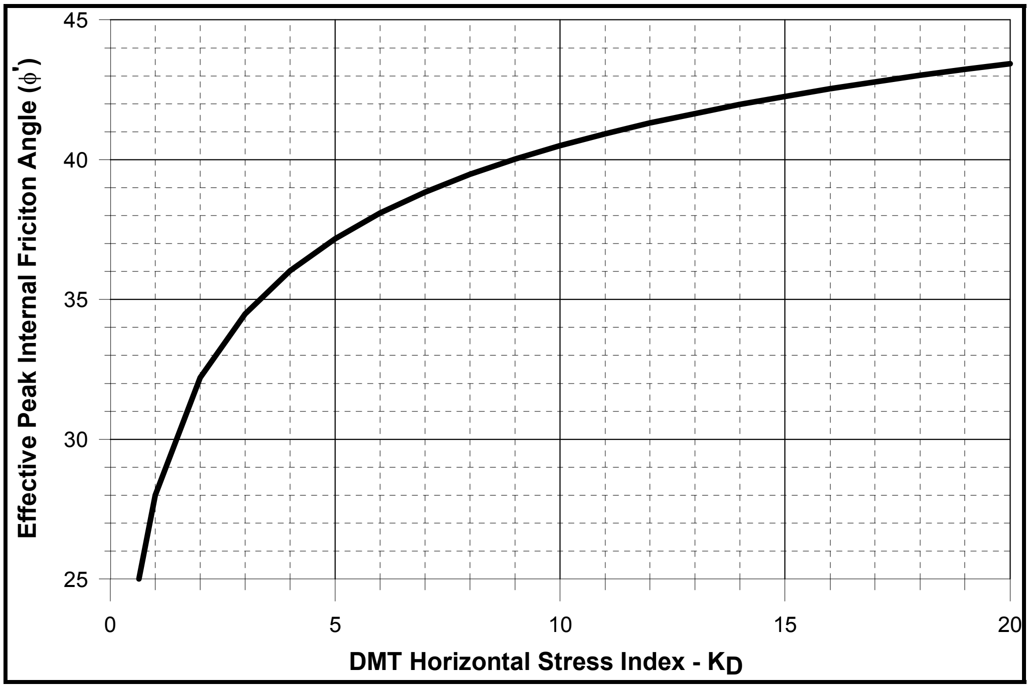


Figure 7-22, Effective Peak Friction Angle and DMT (K_D) Relationship (Robertson and Campanella, 1991)

7.11.2 Cohesive Soils

Drained shear strengths of cohesive soils (i.e. clay, high plasticity silts and residual soils) should be used when there is relatively no change in pore water pressure ($\Delta u \approx 0$) as a result of soil loading such as static driving loads. Geotechnical analyses for these types of loadings should use drained shear strength parameters based on effective stress analyses. The peak or residual drained soil shear strength parameters can be obtained from consolidated drained (CD) triaxial testing (this test is normally not performed because of the time requirements for testing), or consolidated undrained (CU) triaxial testing with pore pressure measurements. Typically for normally consolidated clays the effective cohesion (c') is negligible and is assumed to be equal to zero ($c' = 0$) and the Mohr-Coulomb shear strength equation for drained shear strength of cohesive soils can be expressed as indicated in the following equation.

$$\tau' = \sigma_v' \tan\phi'$$ Equation 7-44

Typically for overconsolidated clays the effective cohesion is greater than zero with the effective friction angle less than that determined for normally consolidated clays. When the preconsolidation pressure (σ'_p or p'_c) is exceeded the overconsolidated clay becomes normally consolidated (see Figure 7 -23).

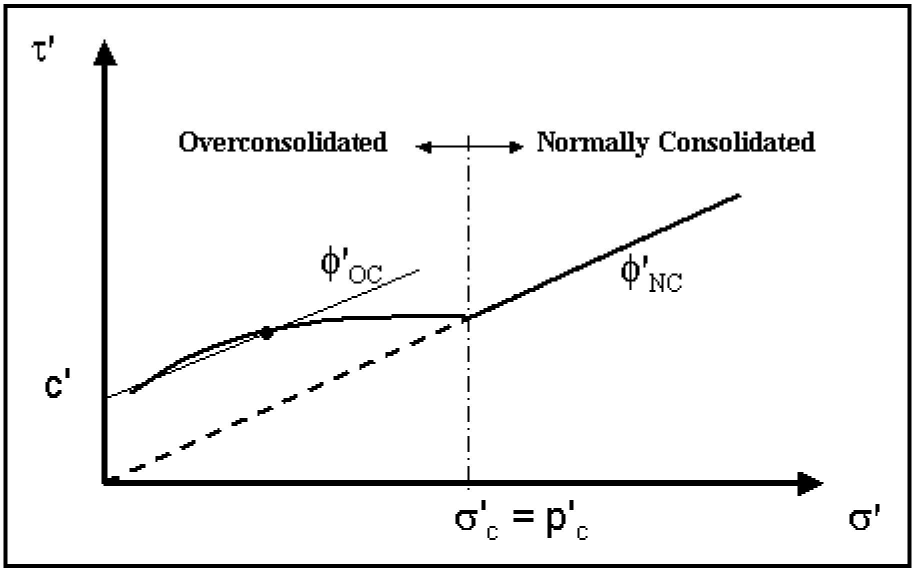


Figure 7-23, Overconsolidated Clay Failure Envelope (CUw/pp Triaxial Test)

The effective peak, fully softened, and residual drained shear strength of cohesive soils should not be evaluated using in-situ testing methods.

Correlations have been developed between drained shear strengths of cohesive soils and index parameters such as plasticity index (I_P or PI), liquid limit (LL), clay fraction (CF) and effective overburden pressure (σ'_{vo} = effective normal stress). Similarly to relationships developed for in-situ testing methods, these relationships for drained shear strengths of cohesive soils were developed based on a soil database that is typically material or soil formation specific and may require verification by laboratory triaxial testing depending on a sensitivity analysis of the impact of the variability of results on the analyses and consequently the impact on the project. These relationships should be used to evaluate the validity of laboratory testing results and to improve the relationship database for regional soil deposits by the SCDOT.

In normally consolidated clays (OCR = 1) the shear strength test will result in a peak effective friction angle (ϕ'). Terzaghi et al. (1996) proposed the relationship in Figure 7-24 between peak effective friction angle (ϕ') for normally consolidated clays and the plasticity index (I_P or PI). For plasticity indices above 60 percent, the peak effective friction angle (ϕ') should be determined from laboratory testing. The Terzaghi et al. (1996) relationship between peak effective friction angle (ϕ') for normally consolidated clays and the plasticity index (I_P or PI) may be estimated by the following equation.

$$\phi' = \mathbf{35.7^o - 0.28(PI) + 0.00145(PI)^2 \pm 8^\circ} \qquad \text{Equation 7-45}$$

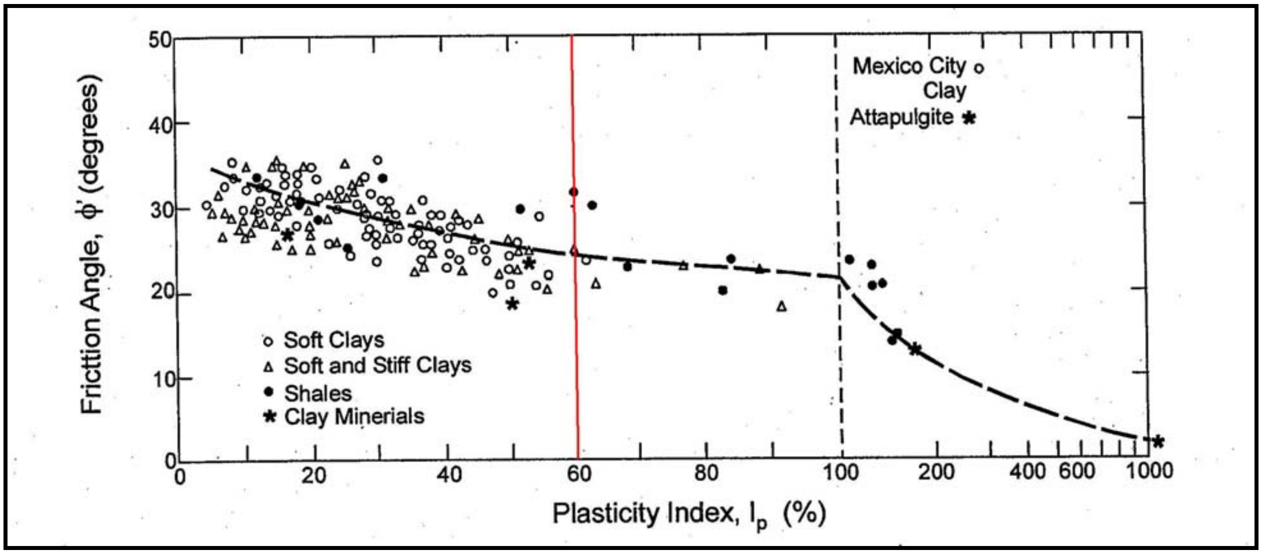


Figure 7-24, Plasticity Index versus Drained Friction Angle For NC Clays (Terzaghi, Peck, and Mesri, 1996)

As indicated earlier, overconsolidated clays reach a peak undrained and then experience shear strain softening to fully softened state. Stark and Eid (1997) proposed the relationship indicated in Figure 7-25 to estimate the fully softened or the peak normally consolidated (NC) effective friction angle (ϕ'). This correlation uses the Liquid Limit (LL), clay size fraction (CF %), and effective overburden pressure (σ'_{vo} = effective normal stress).

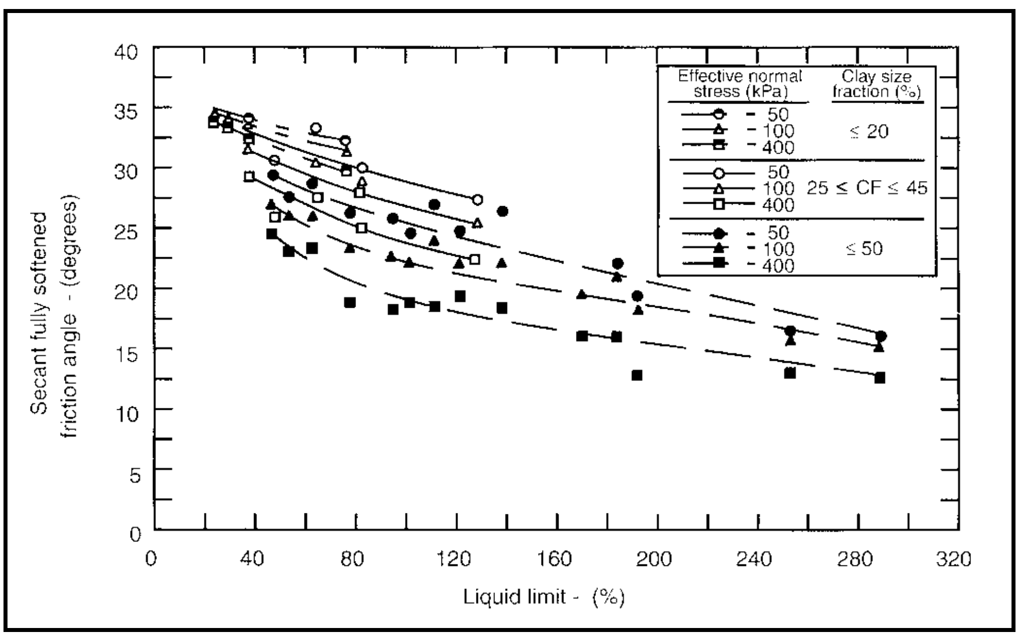


Figure 7-25, Fully Softened (NC) Friction Angle and Liquid Limit Relationship (Stark and Eid, 1997)

For either normally consolidated (OCR = 1) or overconsolidated (OCR > 1) the drained residual friction angle is the same. Stark and Eid (1994) proposed the relationship indicated in Figure 7-26 to estimate the effective residual friction angle (ϕ'_r). This correlation uses the Liquid Limit

(LL), clay size fraction (CF %), and effective overburden pressure (σ'_{vo} = effective normal stress).

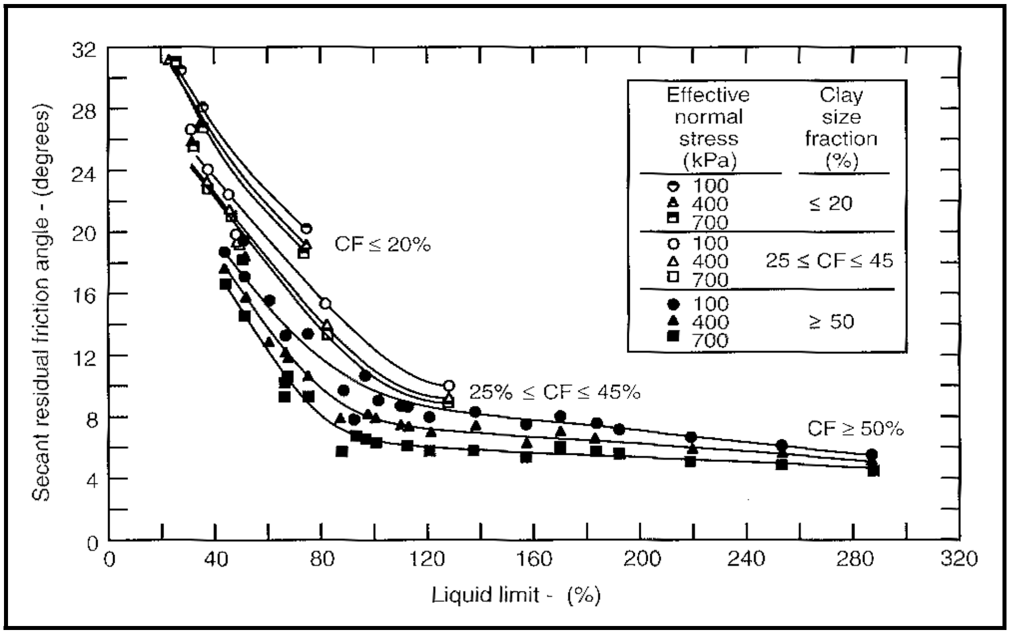


Figure 7-26, Drained Residual Friction Angle and Liquid Limit Relationship (Stark and Eid, 1994)

7.11.3 ϕ' – c' Soils

The drained shear strength of soils that have both ϕ' and c′ components should be determined in the laboratory using the appropriate testing methods. However, if the samples for this type of testing have not been obtained (e.g. during the preliminary exploration), then the soil should be treated as if the soil were either cohesive soils or cohesionless soils. For soils that are difficult to determine the approximate classification, the drained shear strength parameters for both cohesive and cohesionless should be determined and the more conservative design should be used.

7.11.4 Maximum Allowable Effective Soil Shear Strength

SCDOT has established maximum allowable effective soil shear strength design parameters (c', ϕ') shown in Table 7-16, for use in design. These soil shear strength design parameters (c', ϕ') may not be exceeded without laboratory testing and the written permission of the PCS/GDS.

Table 7-16, Maximum Allowable Effective Soil Shear Strengths

Soil Description		Peak [1]		Residual	
USCS	Description	c' (psf)	ϕ' (degrees)	c' (psf)	ϕ' (degrees)
GW, GP, GM, GC	Stone and Gravel	0	40	0	34
SW	Coarse Grained Sand	0	38	0	32
SM, SP	Fine Grained Sand	0	36	0	30
SP	Uniform Rounded Sand	0	32	0	32
ML, MH, SC	Silt, Clayey Sand, Clayey Silt	0	30	0	27
SM-ML	Residual Soils	0	27	0	22
CL-ML	NC Clay (Low Plasticity)	0	35	0	31
CL, CH	NC Clay (Med-High Plasticity)	0	26	0	16
CL-ML	OC Clay (Low Plasticity)	0	34	0	31
CL, CH	OC Clay (Med-High Plasticity)	0	28	0	16

(1) The same maximum peak effective shear strength parameters shall be used for peak effective internal friction angle of normally consolidated cohesive soils and to the fully-softened internal friction angle of overconsolidated cohesive soils.

7.12 BORROW MATERIALS SOIL SHEAR STRENGTH SELECTION

This section pertains to the selection of soil shear strength design parameters for borrow materials used in embankments or behind retaining walls (other than MSE walls or reinforced slopes). Soil shear strength selection shall be based on the soil loading and soil response considerations presented in Section 7.9. The soil shear strength design parameters selected must be locally available, cost effective, and be achievable during construction. The selection of soil shear strength design parameters that require the importation of materials from outside of the general project area should be avoided. To this end, bulk samples will be obtained from existing fill embankments or from proposed cut areas tested as indicated in Chapter 4. The purpose of sampling and testing the existing fill is the assumption that similar fill materials will be available locally. The purpose of sampling and testing proposed cut areas is to determine the suitability of the material for use as fill. The selection of soil shear strength design required for borrow sources should take into consideration the construction borrow specifications as indicated in Section 7.12.1.

The procedure for selecting soil shear strength design parameters varies depending on the type of project as indicated below:

- **<u>Design-Build Projects:</u>** The selection of soil shear strength design parameters for borrow materials requires that the Contractor obtain soil shear strength parameters from all potential borrow pit sources. Evaluation of the soil shear strength design parameters requires that a composite bulk sample be obtained from the borrow source and have the following laboratory tests performed:
    - Moisture Density Relationship (Standard Proctor)
    - Grain Size Distribution with wash #200 Sieve
    - Moisture-Plasticity Relationship Determination (Atterberg Limits)
    - Natural Moisture Content

- Consolidated Undrained (CU) Triaxial Shear Test with pore pressure measurements (sample remolded to 95% of Standard Proctor with moisture -1 percent to +2 percent of optimum moisture content) to obtain drained and undrained shear strength parameters

- **<u>Traditional Design-Bid-Build W/Existing Embankments:</u>** This type of project can occur when existing roads are being improved by widening the existing road. An investigation of locally available materials should be made to confirm that the existing embankment soils are still locally available. If the existing embankment soils are available, the selection of soil shear strength design parameters for these type of projects will be based on using laboratory testing from composite bulk sample obtained from the existing embankment as required in Chapter 4 and appropriately select the drained and undrained soil shear strength design parameters for the borrow material. The plans and contract documents may specify the minimum required soil shear strength parameters for the borrow sources based on the existing embankment soils, if necessary. If the existing embankment soils are not locally available, the borrow material shear strength parameters will be determined as if the project were on a new alignment.

- **<u>Traditional Design-Bid-Build On New Alignment:</u>** This type of project requires the pre-selection of soil shear strength design parameters without performing any laboratory testing. The preliminary subsurface investigation may need to identify locally available soils (or borrow sources) and appropriately select soil shear strength design parameters for the borrow materials. Locally available soils can be investigated by using USDA Soil Survey maps as indicated in Section 7.12.2. The plans and contract documents may specify the minimum required soil shear strength parameters for the borrow sources, if necessary.

7.12.1 <u>SCDOT Borrow Specifications</u>

The 2007 SCDOT Standard Specifications For Highway Construction, Section 203, provides the requirements for borrow material. Embankment material must not have optimum moisture content greater than 25.0% as defined in accordance with SC-T-29. Acceptable soils for use in embankments and as subgrade vary by county indicated by the following two Groups.

- **<u>Group A:</u>** Includes the following counties: Abbeville, Anderson, Cherokee, Chester, Edgefield, Fairfield, Greenville, Greenwood, Lancaster, Laurens, McCormick, Newberry, Oconee, Pickens, Saluda, Spartanburg, Union, and York. Below the upper 5 feet of embankment, any soil that does not meet the description of muck may be used provided it is stable when compacted to the required density.

- **<u>Group B:</u>** Aiken, Allendale, Bamberg, Barnwell, Beaufort, Berkeley, Calhoun, Charleston, Chesterfield, Clarendon, Colleton, Darlington, Dillon, Dorchester, Florence, Georgetown, Hampton, Horry, Jasper, Kershaw, Lee, Lexington, Marion, Marlboro, Orangeburg, Richland, Sumter, and Williamsburg. The soil material below the upper 5 feet of embankment is soils that classify as A-1, A-2, A-3, A-4, A-5, and A-6.

Groups A and B are shown graphically on a South Carolina map in Figure 7-27.

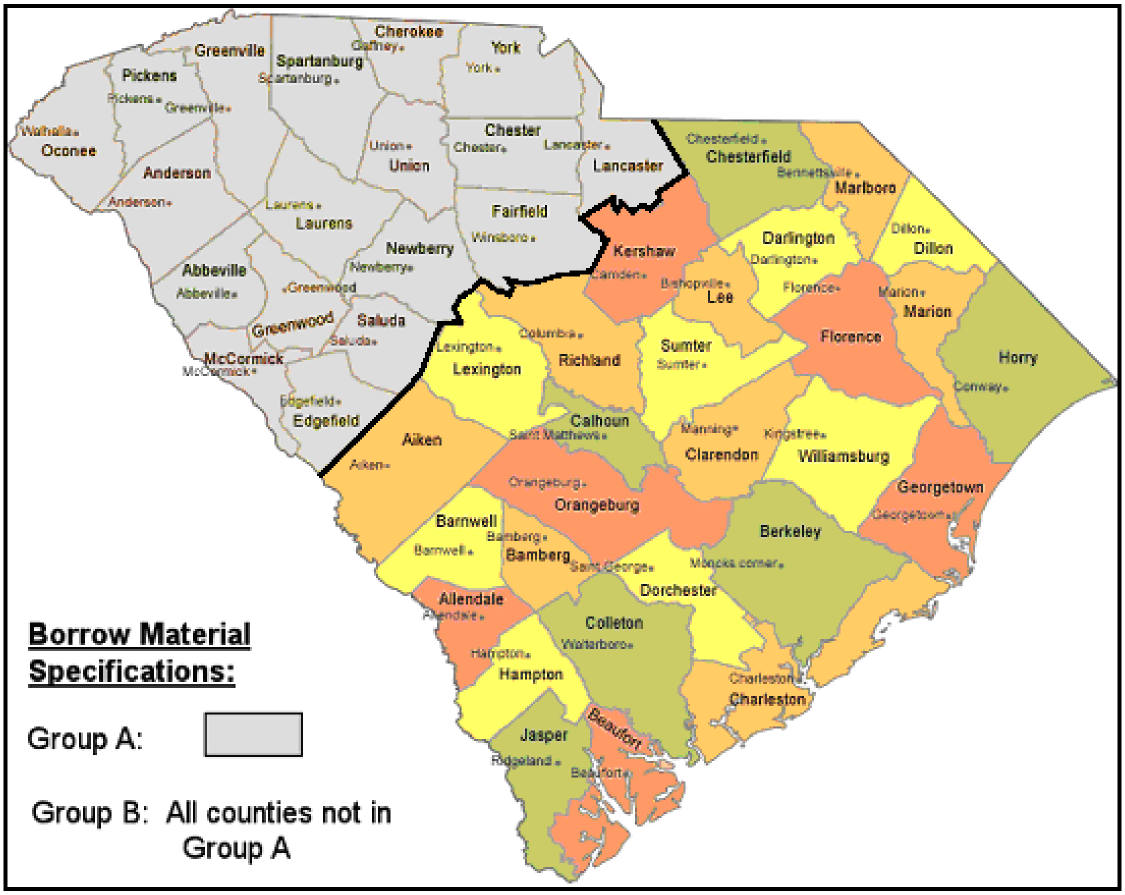


Figure 7-27, Borrow Material Specifications By County

A brief geologic description of the surface soils in Groups A and B are provided below and for more detail see Chapter 11.

- **<u>Group A</u>**: This group is located northwest of the "Fall Line" in the Blue Ridge and Piedmont physiographic geologic units. The Blue Ridge unit surface soils typically consist of residual soil profile consisting of clayey soils near the surface where weathering is more advanced, underlain by sandy silts and silty sands. There may be colluvial (old land-slide) material on the slopes. The Piedmont unit has a residual soil profile that typically consists of clayey soils near the surface, where soil weathering is more advanced, underlain by sandy silts and silty sands. The residual soil profile exists in areas not disturbed by erosion or the activities of man.

- **<u>Group B:</u>** This group is located south and east of the "Fall Line" in the Coastal Plain physiographic geologic unit. Sedimentary soils are found at the surface that consist of unconsolidated sand, clay, gravel, marl, cemented sands, and limestone.

7.12.2 USDA Soil Survey Maps

Locally available borrow sources can be researched by using the United States Department of Agriculture (USDA) Soil Survey Maps. A listing of USDA Soil Surveys that are available can be obtained by selecting "South Carolina" at http://soils.usda.gov/survey/printed_surveys/ and reviewing results by county.

Soil surveys can be obtained as either printed documents, CD-ROM, downloading online .pdf documents, or generated using USDA Web Soil Survey (WWS) Internet application.

The USDA Soil Survey Maps typically indicate Soil Map Units that are described based on USDA textural classification system. Recent USDA Soil Surveys manuscripts contain tables with equivalent material descriptions for the AASHTO soil classification system and the Unified Soil Classification system (USCS). When only the USDA textural classification is indicated in the maps, the geotechnical engineer will need to correlate the USDA textural classifications to the AASHTO soil classification system and USCS.

The USDA Web Soil Survey (WSS) Internet application can be accessed at: http://websoilsurvey.nrcs.usda.gov/app/. The USDA Web Soil Survey (WSS) is an online web application that can provide soil data and natural resource information produced by the National Cooperative Soil Survey. The web site is under constant development and being updated with new information. Soil survey maps and maps of Roadfill sources for project specific locations can be generated as shown in Figure 7-28 and Figures 7-29, respectively.

Figure 7-28, USDA Soil Map – Newberry County, South Carolina (USDA Web Soil Survey - WSS)

Figure 7-29, USDA Roadfill Source Map - Newberry County, South Carolina (USDA Web Soil Survey - WSS)

7.12.3 Compacted Soils Shear Strength Selection

Compacted soils are used to construct roadway embankments, bridge approaches, and backfill behind retaining walls. This Section does not govern the selection of backfill soil properties for MSE walls or reinforced slopes. The method of selecting soil shear strength parameters for compacted soils will be either:

- Measured using consolidated-undrained triaxial tests with pore pressure measurements.
- Conservatively selected based on drained soil shear strength parameters typically encountered in South Carolina soils.

The method to be used for selection will be dependent on the type of project as discussed in Section 7.12.

SCDOT experience with borrow materials typically found in Group A are Piedmont residual soils. These borrow materials are typically classified as micaceous clayey silts and micaceous sandy silts, clays, and silty soils in partially drained conditions. These soils may have USCS classifications of either ML or MH and typically have liquid limits (LL) greater than 30. Published laboratory shear strength testing results for Piedmont residual soils (Sabatini, 2002, Appendix A, page A-40) indicate an average effective friction angle of 35.2° with a ± 1 standard deviation range of $29.9^{\circ} < \phi' < 40.5^{\circ}$. A conservative lower bound of 27.3° is also indicated.

SCDOT experience with borrow materials typically found in Group B are Coastal Plain soils that are typically uniform fine sands that are sometimes difficult to compact and behave similar to silts. When these soils are encountered, caution should be used in selecting effective soil shear strength friction angles since values typically range from $28^{\circ} < \phi' < 32^{\circ}$.

7.12.4 Maximum Allowable Soil Shear Strengths Compacted Soils

Maximum acceptable effective soil shear strength parameters (***c'*, *ϕ'***) have been established in Table 7-18. Maximum total shear strength parameters for cohesive soils is 1,500 psf for CL-ML and 2,500 psf for CL and CH. Values outside of these ranges may only be used if the specific source of material is identified for the project and enough material is available for construction. The selection prior to or during design of a specific source of material is anticipated to occur only during design/build projects. A request for exceeding the stated maximums must be made in writing to the PCS/GDS. The PCS/GDS will indicate what testing is required prior to acceptance of exceeding the maximums. Upon receipt of the testing results, the PCS/GDS shall issue a letter to the project team indicating acceptance or rejection of the request for exceeding the range of acceptable range of soil shear strengths.

Table 7-17, Maximum Allowable Soil Shear Strengths For Compacted Soils

Soil Description		Effective	
USCS	Description	*c'* (psf)	*ϕ'* (degrees)
GW, GP, GM, GC	Stone and Gravel	0	38
SW	Coarse Grained Sand	0	36
SM, SP	Fine Grained Sand	0	34
SP	Uniform Rounded Sand	0	30
ML, MH, SC	Silt, Clayey Sand, Clayey Silt	50	28
SM-ML	Residual Soil	50	24
CL-ML	Clay (Low Plasticity)	50	32
CL, CH	Clay (Med-High Plasticity)	50	26

7.13 SOIL SETTLEMENT PARAMETERS

Settlements are caused by the introduction of loads (stresses) on to the subsurface soils located beneath a site. These settlements can be divided into two primary categories, elastic and time-dependent settlements (consolidation). Settlements (strains) are a function of the load (stress) placed on the subsurface soils. Elastic settlements typically predominate in the cohesionless soils while time-dependent settlements predominate in cohesive soils. Settlement parameters can be developed from high quality laboratory testing (triaxial shear for elastic parameters and consolidation testing for time-dependent parameters). However, for cohesionless soils, obtaining high quality samples for testing can be extremely difficult. Therefore, in-direct methods (correlations) of measuring the elastic parameters are used. Time-dependent settlement parameters correlations for cohesive soils also exist. These correlations should be used for either preliminary analyses or for evaluating the accuracy of laboratory consolidation testing.

7.13.1 Elastic Parameters

Elastic settlements are instantaneous and recoverable. These settlements are calculated using elastic theory. The determination of elastic settlements is provided in Chapter 17. In the determination of the elastic settlements the elastic modulus, E, (tangent or secant) and the Poisson's ratio, ν, are used. Since E and ν are both dependent of the laboratory testing method (unconfined, confined, undrained, drained), the overconsolidation ratio, water content, strain rate and sample disturbance, considerable engineering judgment is required to obtain reasonable values for use in design. Provided in Table 7-19 are elastic modulus correlations with $N^*_{1,60}$ values. Table 7-20 provides typical values of soil elastic modulus and Poisson's ratio for various soil types.

Table 7-18, Elastic Modulus Correlations For Soil (AASHTO, 2007)

Soil Type	Elastic Modulus, E_s (psi)
Silts, sandy silts, slightly cohesive mixtures	$56N^*_{1,60}$
Clean fine to medium sands and slightly silty sands	$97N^*_{1,60}$
Coarse sands	$139N^*_{1,60}$
Sandy gravels and gravels	$167N^*_{1,60}$

Table 7-19, Typical Elastic Modulus and Poisson Ratio Values For Soil (AASHTO, 2007)

Soil Type	Typical Elastic Modulus Values, E (ksi)	Poisson's Ratio, ν
Clay:		0.4 – 0.5 (Undrained)
Soft sensitive	0.347 – 2.08	
Medium stiff to stiff	2.08 – 6.94	
Very stiff	6.94 – 13.89	
Silt	0.278 – 2.78	0.3 – 0.35
Fine Sand:		0.25
Loose	1.11 – 1.67	
Medium dense	1.67 – 2.78	
Dense	2.78 – 4.17	
Sand:		
Loose	1.39 – 4.17	0.20 – 0.36
Medium dense	4.17 – 6.94	
Dense	6.94 – 11.11	0.30 – 0.40
Gravel:		
Loose	4.17 – 11.11	0.20 – 0.35
Medium dense	11.11 – 13.89	
Dense	13.89 – 27.78	0.30 – 0.40

7.13.2 Consolidation Parameters

Consolidation settlements involve the removal of water from the interstitial spaces between soil grains and the rearrangement of the soil grains. Typically, fine-grained soils (clays and silts) are considered to undergo consolidation settlements. However, sands and gravels may also undergo consolidation settlements. The consolidation settlements in sands and gravels occur very quickly, usually during construction, because of the relatively pervious nature of these materials. Fine-grained soils are typically more impervious and therefore will require more time to settle. Further these soil types may also undergo more settlement than sands and gravels because of the volume of water within these soils. To determine the amount of consolidation settlement that a soil will undergo, the following soil parameters are required: compression, recompression, and secondary compression indices, consolidation coefficient and the preconsolidation pressure. These parameters are normally determined from consolidation testing (see Chapter 5). However, for preliminary estimates and to verify the results of the consolidation testing the correlations listed in the following sections may be used. These correlations should not be used for final design, except where the geotechnical design engineer considers the results of the consolidation testing to be questionable. The engineer shall document the reason for the use of the correlations. In addition, all of the consolidation parameters shall be clearly provided in the geotechnical report.

7.13.2.1 Compression Index

The Compression Index (C_c) has been related to Atterberg Limits by Terzaghi and Peck (1967) and Wroth and Wood (1978). The Terzaghi and Peck formula (Equation 7-46) is limited to inorganic clays with sensitivity up to 4 and a LL less than 100. In addition, NAVFAC (1982) (Equations 7-47 and 7-48) also provides a correlation between C_c and e_o that is applicable to all clays.

$$\mathbf{C_c = 0.009(LL - 10)} \qquad \text{Equation 7-46}$$

$$\mathbf{C_c = 0.5G_s\left(\frac{PI}{100}\right)} \qquad \text{Equation 7-47}$$

$$\mathbf{C_c = 1.15(e_o - 0.35)} \qquad \text{Equation 7-48}$$

Where,

LL = Liquid Limit (%)
PI = Plasticity Index (%)
G_S = Specific gravity of the solids
e_o = initial void ratio

The Compression Index may also be related to strain as indicated below.

$$C_{\varepsilon c} = \frac{C_c}{(1 + e_o)} \qquad \text{Equation 7-49}$$

7.13.2.2 Recompression Index

The Recompression Index (C_r) can be correlated to the C_c values. Ladd (1973) indicates the C_r value is approximately 10 to 20 percent of the C_c value. The Recompression Index may also be related to strain as indicated by the following equation.

$$C_{\varepsilon r} = \frac{C_r}{(1 + e_o)} \qquad \text{Equation 7-50}$$

7.13.2.3 Secondary Compression Index

Secondary compression occurs after the completion of elastic and primary consolidation settlements. Secondary compression settlement should be included in the estimate of total settlement for a given project. The amount of secondary compression settlement should be determined. The Secondary Compression Index (C_α) like the other consolidation settlement parameters is best determined from consolidation testing; however, correlations exist that may be used to provide a preliminary estimate of secondary compression settlement. In addition, these correlations may be used to verify the results of the consolidation testing. Provided in Figure 7-30 is a chart of C_α versus the natural moisture content of soil.

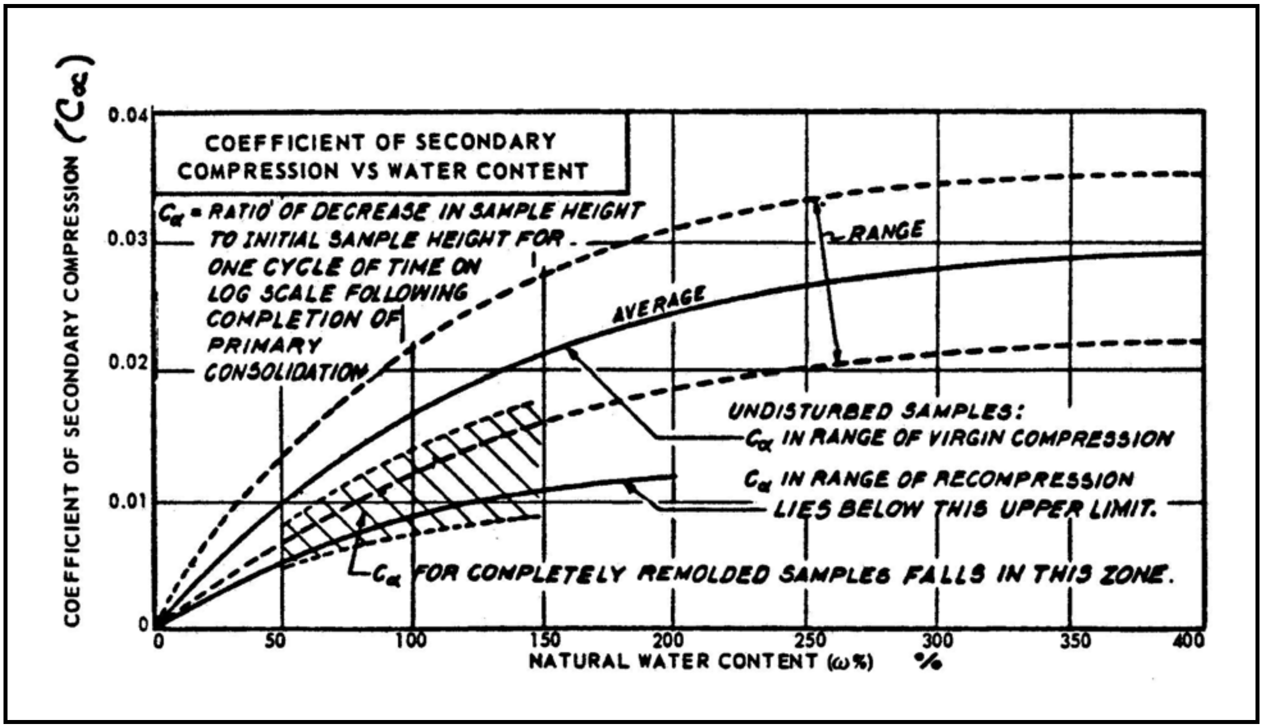


Figure 7-30, Secondary Compression Index Chart
NAVFAC DM-7.1, 1982

The Secondary Compression Index may also be related to strain as indicated below.

$$C_{\varepsilon\alpha} = \frac{C_\alpha}{(1 + e_o)} \qquad \text{Equation 7-51}$$

7.13.2.4 Consolidation Coefficient

The preceding sections dealt with the amount of settlement that could be anticipated at a project location. This section will provide the methods to estimate the time for consolidation settlement. As indicated previously, elastic settlements are anticipated to occur relatively instantaneously (i.e. during construction) while consolidation settlements are anticipated to occur at some time after the structure has been completed. The rate of consolidation is directly related to the permeability of the soil. As with the consolidation parameters, the Consolidation Coefficient (C_v) should be determined from the results of consolidation testing. Correlations exist that may be used to provide a preliminary estimate of Consolidation Coefficient. In addition, these correlations may be used to verify the results of the consolidation testing. Provided in Figure 7-31 is a chart of C_v versus the Liquid Limit of soil.

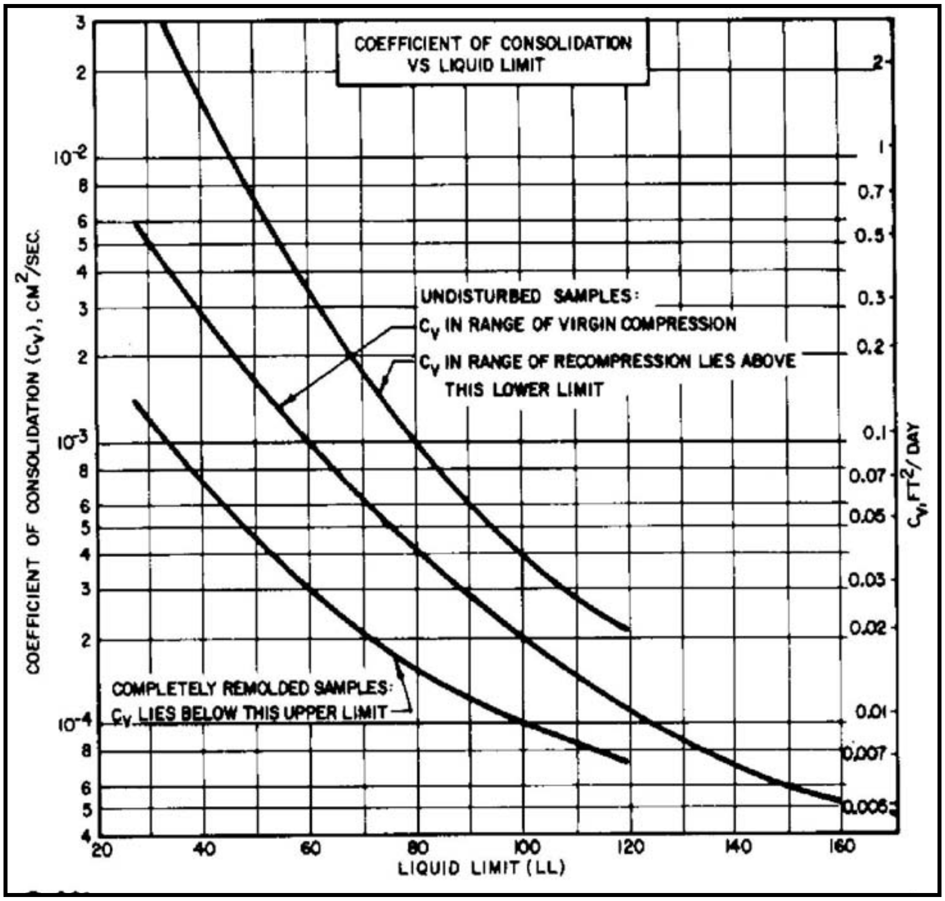


Figure 7-31, Consolidation Coefficient and Liquid Limit Relationship
NAVFAC DM-7.1, 1982

7.13.2.5 Effective Preconsolidation Stress

The effective preconsolidation stress (σ'_p or p'_c) in soils is used to determine whether to use the Compression or Recompression Index. The effective preconsolidation stress (σ'_p) is the maximum past pressure that a soil has been exposed to since deposition. Similarly to the other consolidation parameters the σ'_p is best determined from consolidation testing. Correlations also exist; however, these correlations should only be used for either preliminary analyses or for evaluating the accuracy of laboratory consolidation testing. The effective preconsolidation stress (σ'_p or p'_c) can be correlated to total cohesion, c_u (NAVFAC DM-7.1, 1986). As with the other consolidation parameters the correlated σ'_p should be used for preliminary estimates only.

$$\boldsymbol{\sigma'_p} = \frac{\mathbf{c_u}}{(\mathbf{0.11 + 0.0037PI})}$$ Equation 7-52

The σ'_p can also be estimated from Cone Penetrometer Testing (CPT) using the following equations (Sabatini, 2002).

$$\sigma'_p = 0.33(q_c - \sigma_v)$$ Equation 7-53

CPT Piezocone (face element):

$$\sigma'_p = 0.47(u_1 - u_o)$$ Equation 7-54

CPT Piezocone (shoulder element):

$$\sigma'_p = 0.54(u_2 - u_o)$$ Equation 7-55

7.14 ROCK PARAMETER DETERMINATION

While the shear strength of individual rock cores is obtained from unconfined axial compression testing, the shear strength of the entire rock mass should be used for design. Therefore, the shear strength and consolidation parameters should be developed using the RMR as defined in Chapter 6.

7.14.1 Shear Strength Parameters

The rock mass shear strength should be evaluated using the Hoek and Brown criteria (AASHTO, 2007). The shear strength of the rock mass is represented by a curved envelope that is a function of the unconfined (uniaxial) compressive strength of the intact rock, q_u, and two dimensionless factors. The rock mass shear strength, τ, (in ksf) is defined as indicated below.

$$\tau = (\cot\phi'_i - \cos\phi'_i)\, m \left(\frac{q_u}{8}\right)$$ Equation 7-56

$$\phi'_i = \tan^{-1}\left\{4h\cos^2\left[30 + 0.33\sin^{-1}\left(h^{-1.5}\right)\right] - 1\right\}^{-0.5}$$ Equation 7-57

$$h = 1 + \frac{\left[16\left(m\,\sigma'_n + s\,q_u\right)\right]}{3\,m^2 q_u}$$ Equation 7-58

Where,

ϕ'_I = instantaneous friction angle of the rock mass (degrees)
q_u = average unconfined rock core compressive strength (ksf)
σ'_n = effective normal stress (ksf)
m and s from Table 7-21

Table 7-20, Constants m and s based on RMR (AASHTO, 2007)

Rock Quality	Constants	Rock Type: **A** = Carbonate rocks with well developed crystal cleavage – dolomite, limestone and marble **B** = Lithified argrillaceous rocks – mudstone, siltstone, shale and slate (normal to cleavage) **C** = Areanaceous rocks with strong crystals and poorly developed crystal cleavage – sandstone and quartzite **D** = Fine-grained polyminerallic igneous crystalline rocks – andesite, dolerite, diabase and rhyolite **E** = Coarse-grained polyminerallic igneous and metamorphic crystalline rocks – amphilboltie, gabbro, gneiss, granite, norite, and quartz-diorite				
		A	**B**	**C**	**D**	**E**
Intact rock samples RMR = 100	m s	7.00 1.00	10.00 1.00	15.00 1.00	17.00 1.00	25.00 1.00
Very good quality rock mass RMR = 85	m s	2.40 0.082	3.43 0.082	5.14 0.082	5.82 0.082	8.567 0.082
Good quality rock mass RMR = 65	m s	0.575 0.00293	0.821 0.00293	1.231 0.00293	1.395 0.00293	2.052 0.00293
Fair quality rock mass RMR = 44	m s	0.128 0.00009	0.183 0.00009	0.275 0.00009	0.311 0.00009	0.458 0.00009
Poor quality rock mass RMR = 23	m s	0.029 $3*10^{-6}$	0.041 $3*10^{-6}$	0.061 $3*10^{-6}$	0.069 $3*10^{-6}$	0.102 $3*10^{-6}$
Very poor quality rock mass RMR = 3	m s	0.007 $1*10^{-7}$	0.010 $1*10^{-7}$	0.015 $1*10^{-7}$	0.017 $1*10^{-7}$	0.025 $1*10^{-7}$

7.14.2 Elastic Parameters

Rocks will primarily undergo elastic settlements. The elastic settlements will be instantaneous and recoverable. These settlements are calculated using elastic theory. The determination of elastic settlements is provided in Chapter 17. In the determination of the elastic settlements, the elastic modulus, E, is required. The elastic modulus of a rock mass is the lesser of modulus determined from intact rock core testing or from the equations below (AASHTO, 2007).

$$E_m = 145\left(10^{\left(\frac{RMR-10}{40}\right)}\right) \qquad \text{Equation 7-59}$$

$$E_m = \left(\frac{E_m}{E_i}\right)E_i \qquad \text{Equation 7-60}$$

Where,

E_m = elastic modulus of rock mass (ksi)
E_i = elastic modulus of intact rock (ksi)
RMR = Adjusted Rock Mass Rating from Chapter 6

7.15 REFERENCES

AASHTO LRFD Bridge Design Specifications Customary U.S. Units, 4th Edition, dated 2007. American Association of State Highway and Transportation Officials, Washington, D.C.

Aas, G., S. Lacasse, I, Lunne, and K. Hoek, (1986), *Use of In-Situ Tests for Foundation Designs in Clay*, In Situ '86 Proceedings, ASCE.

Cetin, K. O., R. B. Seed, A. Der Kiureghian, K. Tokimatsu, L. F. Harder, R. E. Kayen and R. E. S. Moss, (2004) *Standard Penetration Test-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential*, Journal of Geotechnical and Geoenvironmental Engineering, ASCE.

Duncan, J. M. and A. L. Buchignani (1976), An Engineering Manual for Settlement Studies, Virginia Polytechnic Institute and State University.

Duncan, J. M. and S. G. Wright, (2005), Soil Strength and Slope Stability, John Wiley & Sons, Inc., New Jersey.

Hatanaka, M. and A. Uchida, (1996) *Empirical Correlation Between Penetration Resistance and Internal Friction Angle of Sandy Soils,* Soils and Foundations, Vol. 36, No. 4.

Jamiolkowski, M., C. C. Ladd, J. Germaine and R. Lancellotta, (1985), *New Developments in Field and Lab Testing of Soils*, 11th International Conference on Soil Mechanics and Foundations Engineering Proceedings, Vol. 1.

Jefferies, M. G. and M. P. Davies, (1993), *Use of CPTu to Estimate Equivalent SPT* N_{60}*,* Geotechnical Testing Journal, American Society of Testing Materials.

McGregor, J. A. and J. M. Duncan, (1998) Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice, Virginia Polytechnic Institute and State University.

Mitchell, J. K. Fundamentals of Soil Behavior, 2nd Edition, John Wiley and Sons, Inc.

Moss, R. E. S., R. B. Seed, R. E. Kayen, J. P. Stewart, and A. Der Kiureghian, CPT-Based Probabilistic Assessment of Seismic Soil Liquefaction Initiation, (2006), Publication No. PEER 2005/15, Pacific Earthquake Engineering Research Center.

Robertson, P. K., and R. G. Campanella, (1983), *Interpretation of Cone Penetration Tests,* Canadian Geotechnical Journal, Vol. 20, No. 4.

Sabatini, P. J., R. C. Bachus, P. W. Mayne, J. A. Schneider and T. E. Zettler, (2002), Geotechnical Engineering Circular No. 5 – Evaluation of Soil and Rock Properties, (Publication No. FHWA-IF-02-034). US Department of Transportation, Office of Bridge Technology, Federal Highway Administration, Washington, D.C.

Soils and Foundation Workshop Reference Manual, dated July 2000 (Publication No. FHWA-NHI-00-045). National Highway Institute, US Department of Transportation, Federal Highway Administration, Washington D.C.

Soil Mechanics – Design Manual 7.1, dated May 1982 (Publication No. NAVFAC DM-7.1). Department of the Navy, Naval Facilities Engineering Command, Alexandria, Virginia.

Subsurface Investigations, dated March 1997 (Publication No. FHWA-HI-97-021). National Highway Institute, US Department of Transportation, Federal Highway Administration, Washington D.C.

Terzaghi, K., R. B. Peck, and G. Mesri, (1996), Soil Mechanics In Engineering Practice, John Wiley & Sons, Inc., Third Edition, New York.

U.S. Department of Interior, Bureau of Reclamation (1998), “Earth Manual – Part I”, Third Edition Earth Sciences and Research Laboratory Geotechnical Research Technical Service Center Denver, Colorado.

Chapter 8

GEOTECHNICAL

LRFD DESIGN

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Table of Contents

Section **Page**

List of Tables

Title **Page**

CHAPTER 8

GEOTECHNICAL LRFD DESIGN

8.1 INTRODUCTION

Geotechnical engineering analyses and designs for transportation structures have traditionally been based on Allowable Stress Design (ASD), also known as Working Stress Design (WSD). Transportation structures that require geotechnical engineering are bridge foundations, sign and lighting foundations, earth retaining structures (MSE walls, reinforced concrete walls, brick walls, cantilever walls, etc.), and roadway embankments (at bridge approaches and along roadways). The primary guidance for the ASD design methodology has been the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges (17th edition – last edition published 2002) and various Federal Highway Administration (FHWA) geotechnical engineering publications. The ASD methodology is based on limiting the stresses induced by the applied loads (Q, which includes dead loads - DL and live loads - LL) on a component/member from exceeding the allowable (or working) stress of the material (R_{all}). The allowable stress of a material is computed by dividing the nominal strength of the material (R_n) by an appropriate factor of safety (FS) as indicated in the following equation.

$$\mathbf{Q} = \sum \mathbf{DL} + \sum \mathbf{LL} \leq \mathbf{R_{all}} = \frac{\mathbf{R_n}}{\mathbf{FS}} \qquad \text{Equation 8-1}$$

This design approach uses a single factor of safety to account for all of the geotechnical engineering uncertainties. The ASD factors of safety do not appropriately take into account variability associated with the predictive accuracy of dead loads, live loads, wind loads, and earthquake loads or the different levels of uncertainty associated with design methodology, material properties, site variability, material sampling, and material testing. The assignment of ASD factors of safety has traditionally been based on experience and judgment. This methodology does not permit a consistent or rational method of accessing risk.

In 1986 an NCHRP study (20-7/31) concluded that the AASHTO Standard Specifications for Highway Bridges contained gaps and inconsistencies, and did not use the latest design philosophy and knowledge. In response, AASHTO adopted the Load and Resistance Factor Design (LRFD) Bridge Design Specification in 1994 and the Load and Resistance Factor Rating (LRFR) Guide Specification in 2002. The current AASHTO LRFD design specification incorporates state-of-the-art analysis and design methodologies with load and resistance factors based on the known variability of applied loads and material properties. These load and resistance factors are calibrated from actual statistics to ensure a uniform level of safety. Because of LRFD's impact on the safety, reliability, and serviceability of the Nation's bridge inventory, AASHTO, in concurrence with the Federal Highway Administration (FHWA), set a transition deadline of 2007 for bridges and 2010 for culverts, retaining walls and other miscellaneous structures. After this date, States must design all new structures in accordance with the LRFD specifications.

The SCDOT is committed to using the LRFD design methodology on structures including all aspects of geotechnical engineering analysis and design. In this Manual the term AASHTO specifications refers to the AASHTO LRFD Bridge Specifications (latest edition), unless indicated otherwise. The LRFD geotechnical design approach is presented in Chapters 8, 9, and 10 of this Manual. All tables in this Chapter have been modified and adapted from AASHTO specifications unless indicated otherwise. The geotechnical design methodology presented in this Manual provides guidance on how to apply the LRFD geotechnical design approach into geotechnical engineering analyses for SCDOT projects.

8.2 LRFD DESIGN PHILOSOPHY

Basic to all good engineering design methodologies (including the ASD method) with respect to structural or geotechnical engineering is that when a certain Load (*Q* or Demand) is placed on a component/member, there is sufficient Resistance (*R* or Supply) to insure that an established performance criterion is not exceeded as illustrated by the following equation:

$$\textbf{Load } (\boldsymbol{Q}) < \textbf{RESISTANCE } (\boldsymbol{R}) \qquad \text{Equation 8-2}$$

The Load and Resistance quantities can be expressed as a force, stress, strain, displacement, number of cycles, temperature, or some other parameter that results in structural or performance failure of a component/member. The level of inequality between the Load and Resistance side of Equation 8-2 represents the uncertainty. In order to have an acceptable design the uncertainties must be mitigated by applying an appropriate margin of safety in the design.

The LRFD design methodology mitigates the uncertainties by applying individual load factors (γ) and a load modifier (η) to each type of load (Q_i). On the resistance side of the equation a resistance factor (φ) is applied to the nominal resistance (R_n). The sum of the factored loads, Q, placed on the component/member must not exceed the factored resistance of the component/member in order to have satisfactory performance. The following equation illustrates the basic LRFD design concept.

$$\boldsymbol{Q} = \sum \eta_i \gamma_i \boldsymbol{Q_i} \leq \varphi \boldsymbol{R_n} = \boldsymbol{R_r} \qquad \text{Equation 8-3}$$

Where,

- Q = Factored Load
- Q_i = Force Effect
- η_i = Load modifier
- γ_i = Load factor
- R_r = Factored Resistance
- R_n = Nominal Resistance (i.e. ultimate capacity)
- φ = Resistance Factor

Equation 8-3 is applicable to more than one load combination as defined by the condition that defines the “Limit State”.

8.3 LIMIT STATES

A “Limit State” is a condition beyond which a component/member of a foundation or other structure ceases to satisfy the provisions for which the component/member was designed. AASHTO has defined the following limit states for use in design:

- Strength Limit State
- Service Limit State
- Extreme Event Limit State
- Fatigue Limit State

The Fatigue Limit State is the only limit state that is not used in geotechnical analyses or design. A description of the limit states that are used in geotechnical engineering are provided in the following table.

Table 8-1, Limit States
(Modified from FHWA-NHI-05-094)

Limit State	Description
Strength	The strength limit state is a design boundary condition considered to ensure that strength and stability are provided to resist specified load combinations, and avoid the total or partial collapse of the structure. Examples of strength limit states in geotechnical engineering include bearing failure, sliding, and earth loadings for structural analysis.
Service	The service limit state represents a design boundary condition for structure performance under intended service loads, and accounts for some acceptable measure of structure movement throughout the structure’s performance life. Examples include vertical settlement of a foundation or lateral displacement of a retaining wall. Another example of a service limit state condition is the rotation of a rocker bearing on an abutment caused by instability of the earth slope that supports the abutment.
Extreme Event	Evaluation of a structural member/component at the extreme event limit state considers a loading combination that represents an excessive or infrequent design boundary condition. Such conditions may include ship impacts, vehicle impact, and seismic events. Because the probability of these events occurring during the life of the structure is relatively small, a smaller margin of safety is appropriate when evaluating this limit state.

8.4 TYPES OF LOADS

AASHTO specifications classify loads as either permanent loads or transient loads. Permanent loads are present for the life of the structure and do not change over time. Permanent loads are generally very predictable. The following is a list of all loads identified by AASHTO specifications as permanent loads:

- Dead Load of Components – DC
- Downdrag – DD
- Dead Load of Wearing Surface and Utilities – DW
- Horizontal Earth Pressures – EH
- Locked-In Erection Stresses – EL
- Vertical Earth Pressure – EV
- Earth Load Surcharge – ES

A brief description for each of these permanent loads is provided in Table 8-2. For a complete description and method of computing these loads see the AASHTO specifications.

Table 8-2, Permanent Load Descriptions
(Modified from FHWA-NHI-05-094)

AASHTO Designation	Definition	Description
DC	Dead load of structural components and nonstructural attachments	The DC loads include the weight of both fabricated structure components (e.g., structural steel girders and prestressed concrete beams) and cast-in-place structure components (e.g., deck slabs, abutments, and footings). DC loads also include nonstructural attachments such as lighting and signs.
DD	Downdrag	When a deep foundation is installed through a soil layer that is subject to relative settlement of the surrounding soil to the deep foundation, downdrag forces are induced on the deep foundation. The magnitude of DD load may be computed in a similar manner as the positive shaft resistance calculation. Allowance may need to be made for the possible increase in undrained shear strength as consolidation occurs. For the strength limit state, the factored downdrag loads are added to the factored vertical dead load in the assessment of pile capacity. For the service limit state, the downdrag loads are added to the vertical dead load in the assessment of settlement. Downdrag forces can also occur in the Extreme Event I limit state due to downdrag forces resulting from soil liquefaction of loose sandy soil. Measures to mitigate downdrag are typically used by applying a thin coat of bitumen on the deep foundation surface or some other means of reducing surface friction on the pile may reduce downdrag forces.
DW	Dead load of wearing surfaces and utilities	The DW loads include asphalt wearing surfaces, future overlays and planned widening, as well as miscellaneous items (e.g., scuppers, railings and supported utility services).
EH	Horizontal earth pressure load	The EH loads are the force effects of horizontal earth pressures due to partial or full embedment into soil. These horizontal earth pressures are those resulting from static load effects. The magnitude of horizontal earth pressure loads on a substructure are a function of: • Structure type (e.g., gravity, cantilever, anchored, or mechanically-stabilized earth wall) • Type, unit weight, and shear strength of the retained earth • Anticipated or permissible magnitude and direction of horizontal substructure movement • Compaction effort used during placement of soil backfill • Location of the ground water table within the retained soil

Table 8-2 (Continued), Permanent Load Descriptions
(Modified from FHWA-NHI-05-094)

EL	Locked-in erection stresses	The EL loads are accumulated locked-in force effects resulting from the construction process, typically resulting from segmental superstructure construction. These would include precast prestressed or post-tensioned concrete structures. For substructure designs, these force effects are small enough and can be ignored.
EV	Vertical pressure from dead load of earth fill	The vertical pressure of earth fill dead load acts on the top of footings and on the back face of battered wall and abutment stems. The load is determined by multiplying the volume of fill by the density and the gravitational acceleration (unit weight).
ES	Earth surcharge load	The ES loads are the force effects of surcharge loads on the backs of earth retaining structures. These effects must be considered in the design of walls and bridge abutments.

Transient loads may only be present for a short amount of time, may change direction, and are generally less predictable than permanent loads. Transient loads include the following:

- Vehicular braking force - BR
- Vehicular centrifugal force – CE
- Creep - CR
- Vehicular collision force - CT
- Vessel collision force - CV
- Earthquake - EQ
- Friction – FR
- Ice load – IC
- Vehicular dynamic load allowance - IM
- Vehicular live load - LL
- Live load surcharge - LS
- Pedestrian live load - PL
- Settlement - SE
- Shrinkage - SH
- Temperature gradient – TG
- Uniform temperature – TU
- Water load and stream pressure - WA
- Wind on live load - WL
- Wind load on structure - WS

A brief description for each of these transient loads is provided in Table 8-3. For a complete description and method of computing these loads see the AASHTO specifications.

Table 8-3, Transient Load Descriptions
(Modified from FHWA-NHI-05-094)

AASHTO Designation	Definition	Description
BR	Vehicular braking force	The BR loads are the force effects of vehicle braking that is represented as a horizontal force effect along the length of a bridge that is resisted by the structure foundations.
CE	Vehicular centrifugal force	The CE loads are the force effects of vehicles traveling on a bridge located along a horizontal curve and generate a centrifugal force effect that must be considered in design. For substructure design, centrifugal forces represent a horizontal force effect.

Table 8-3 (Continued), Transient Load Descriptions (Modified from FHWA-NHI-05-094)

CR	Creep	These loads are internal force effects that develop on structure components as a result of creep and shrinkage of materials. These forces should be considered for substructure design when applicable.
CT	Vehicular collision force	The CT loads are the force effects of collisions by roadway and rail vehicles.
CV	Vessel collision force	The CV loads are the force effects of vessel collision by ships and barges due to their proximity to navigation waterways. The principal factors affecting the risk and consequences of vessel collisions with substructures in a waterway are related to vessel, waterway, and bridge characteristics.
EQ	Earthquake	**(DO NOT USE *AASHTO* FOR DETERMINATION OF EQ LOADS)** The EQ loads are the earthquake force effects that are predominately horizontal and act through the center of mass of the structure. Because most of the weight of a bridge is in the superstructure, seismic loads are assumed to act through the bridge deck. These loads are due to inertial effects and therefore are proportional to the weight and acceleration of the superstructure. The effects of vertical components of earthquake ground motions are typically small and are usually neglected except for complex bridges. The SCDOT *Seismic Design Specifications for Highway Bridges* specifies two design earthquakes to be used: • *Functional Evaluation Earthquake (FEE).* The ground shaking having a 15% probability of exceedance in 75 years • *Safety Evaluation Earthquake (SEE).* The ground shaking having a 3% probability of exceedance in 75 years For information on how to compute EQ loads for geotechnical earthquake engineering analyses see Chapters 11 and 12 of this Manual and the SCDOT *Seismic Design Specifications for Highway Bridges*.
FR	Friction	Forces due to friction as a result of sliding or rotation of surfaces.
IC	Ice Load	Ice force effects on piers as a result of ice flows, thickness of ice, and geometry of piers. In South Carolina this factor will not be used.
IM	Vehicular dynamic load allowance	The IM loads are the force effects of dynamic vehicle loading on structures. For foundations and abutments supporting bridges, these force effects are incorporated into the loads used for superstructure design. For retaining walls not subject to vertical superstructure reactions and for foundation components completely below ground level, the dynamic load allowance is not applicable.

Table 8-3 (Continued), Transient Load Descriptions (Modified from FHWA-NHI-05-094)

LL	Vehicular live load	The LL loads are the force effects of vehicular live load (truck traffic). The force effects of truck traffic are in part modeled using a highway design "umbrella" vehicle designated HL-93 to represent typical variations in axle loads and spacing. The HL-93 vehicular live load consists of a combination of a design truck HS20-44 and a design lane loading that simulates a truck train combined with a concentrated load to generate a maximum moment or shear effect for the component being designed, and an impact load (not used on lane loadings) to account for the sudden application of the truck loading to the structure.
LS	Live load surcharge	The LS loads are the force effects of traffic loads on backfills that must be considered in the design of walls and abutments. These force effects are considered as an equivalent surcharge. Live load surcharge effects produce a horizontal pressure component on a wall in addition to horizontal earth loads. If traffic is expected within a distance behind a wall equal to about half of the wall height, the live load traffic surcharge is assumed to act on the retained earth surface.
PL	Pedestrian live load	The PL loads are the force effects of pedestrian and/or bicycle traffic loads that are placed on bridge sidewalks or pedestrian bridges.
SE	Settlement	These loads are internal force effects that develop on structure components as a result of differential settlement between substructures and within substructure units.
SH	Shrinkage	These loads are internal force effects that develop on structure components as a result of shrinkage of materials. These forces should be considered for substructure design when applicable.
TG	Temperature gradient	These loads are internal force effects and deformations that develop on structure components as a result of positive and negative temperature gradients with depth in component's cross-section. These forces should be considered for substructure design when applicable.
TU	Uniform temperature	These loads are internal force effects that develop on structure components as a result of thermal movement associated with uniform temperature changes in the materials. These forces should be considered for substructure design when applicable.

Table 8-3 (Continued), Transient Load Descriptions
(Modified from FHWA-NHI-05-094)

WA	Water load and stream pressure	The WA loads are the force effects on structures due to water loading and include static pressure, buoyancy, and stream pressure. Static water and the effects of buoyancy need to be considered whenever substructures are constructed below a temporary or permanent ground water level. Buoyancy effects must be considered during the design of a spread footing or pile cap located below the water elevation. Stream pressure effects include stream currents and waves, and floating debris.
WL	Wind on live load	The WL loads are the wind force effects on live loads. The WL force should only be applied to portions of the structure that add to the force effect being investigated.
WS	Wind load on structure	The WS loads are the wind force effects of horizontal wind pressure on the structure. The effects of vertical wind pressure on the underside of bridges due to an interruption of the horizontal flow of air and the effects of aero-elastic instability represent special load conditions that are typically taken into account for long-span bridges. For small and/or low structures, wind loading does not usually govern the design. However, for large and/or tall bridges, wind loading can govern the design and should be investigated. Where wind loading is important, the wind pressure should be evaluated from two or more different directions for the windward (facing the wind), leeward (facing away from the wind), and side pressures to determine which produce the most critical loads on the structure.

8.5 LOAD COMBINATION LIMIT STATES

The limit states are further subdivided, based on consideration of applicable load. The design of foundations supporting bridge piers or abutments should consider all limit state loading conditions applicable to the structure being designed. A description of the load combination limit states that are used in geotechnical engineering is provided in Table 8-4. Most substructure designs will require the evaluation of foundation and structure performance at the Strength I and Service I limit states. These limit states are generally similar to evaluations of ultimate capacity and deformation behavior in ASD, respectively.

Table 8-4, Load Combination Limit State Considerations (Modified from FHWA-NHI-05-094)

Load Combination Limit State	Load Combination Considerations
Strength I	Basic load combination relating to the normal vehicular use of the bridge without wind.
Strength II	Load combination relating to the use of the bridge by Owner-specified special design vehicles and/or evaluation permit vehicles, without wind.
Strength III	Load combination relating to the bridge exposed to wind velocity exceeding 55 mph without live loads.
Strength IV	Load combination relating to very high dead load to live load force effect ratios exceeding about 7.0 (e.g., for spans greater than 250 ft.).
Strength V	Load combination relating to normal vehicular use of the bridge with wind velocity of 55 mph.
Extreme Event I	Load combination including the effects of a design earthquake.
Extreme Event II	Load combination relating to collision by vessels and vehicles, and certain hydraulic events.
Service I	Load combination relating to the normal operational use of the bridge with 55 mph wind.

8.6 LOAD MODIFIERS

AASHTO LRFD design methodology allows each factored load to be adjusted by a load modifier, η_i. This load modifier, η_i, accounts for the combined effects of ductility, η_D, redundancy, η_R, and operational importance, η_I. In geotechnical design load modifiers are not used to account for the influence of ductility, redundancy, and operational importance on structure performance. The influences of redundancy and operational importance have been incorporated into the selection of the geotechnical resistance factors. Therefore, a load modifier of 1.0 is used by the SCDOT for all geotechnical engineering analyses.

8.7 LOAD COMBINATION AND LOAD FACTORS

Load factors vary for different load types and limit states to reflect either the certainty with which the load can be estimated or the importance of each load category for a particular limit state. Table 8-5 provides load combinations and appropriate load factors to be used on SCDOT geotechnical designs. This table is based on the AASHTO specifications.

These load factors apply only to geotechnical structures. For bridges and roadway structures, the structural designers (Bridge and Roadway Structures) are responsible for evaluating the load combinations and load factors and provide the loads to the geotechnical engineers for analyses. For geotechnical structures where the engineer-of-record is the geotechnical engineer, the geotechnical engineer will be responsible for determining the load combinations and load factors for their geotechnical structure (embankments, MSE walls-external stability, reinforced slopes, etc.).

Table 8-5, Load Combination and Load Factors
(Modified from AASHTO Specifications)

Load Combination Limit State	DC DD DW EH EV ES EL	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH		*Note: Use Only One of These Load Types at a Time*					
							Min	Max	TG	SE	EQ	IC	CT	CV
Strength I	γ_P	1.75	1.00	----	----	1.00	0.50	1.20	γ_{TG}	γ_{SE}	----	----	----	----
Strength II	γ_P	1.35	1.00	----	----	1.00	0.50	1.20	γ_{TG}	γ_{SE}	----	----	----	----
Strength III	γ_P	----	1.00	1.40	----	1.00	0.50	1.20	γ_{TG}	γ_{SE}	----	----	----	----
Strength IV	γ_P	----	1.00	----	----	1.00	0.50	1.20	----	----	----	----	----	----
Strength V	γ_P	1.35	1.00	0.40	1.00	1.00	0.50	1.20	γ_{TG}	γ_{SE}	----	----	----	----
Extreme Event I	γ_P	γ_{EQ}	1.00	----	----	1.00	----	----	----	----	1.00	----	----	----
Extreme Event II	γ_P	0.50	1.00	----	----	1.00	----	----	----	----	----	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.00	1.00	1.00	1.20	γ_{TG}	γ_{SE}	----	----	----	----

The following observations about magnitude and relationship between various load factors indicated in Table 8-5 are listed below:

- A load factor of 1.00 is used for all permanent and most transient loads for Service I.
- The live load factor for Strength I is greater than that for Strength II (i.e., 1.75 versus 1.35) because variability of live load is greater for normal vehicular traffic than for a permit vehicle.
- The live load factor for Strength I is greater than that for Strength V (i.e., 1.75 versus 1.35) because variability of live load is greater for normal vehicular use without wind than for a bridge subjected to a wind of 55 mph, and because less traffic is anticipated during design wind conditions.
- The load factor for wind load on structures for Strength III is greater than for Strength V (i.e., 1.40 versus 0.40) because the wind load represents the primary load for Strength III where structures are subjected to a wind velocity greater than 55 mph, compared to Strength V where wind velocity of 55 mph represents just a component of all loads on the structure.
- The live load factor for Strength III is zero because vehicular traffic is considered unstable and therefore unlikely under extreme wind conditions.
- The load factors for wind load for Strength V are less than 1.00 (i.e., 0.40) to account for the probability of the maximum value of these loads occurring simultaneously.

The load factor temperature gradient (γ_{TG}) shall be selected by the structural designer in accordance with AASHTO specifications or other governing design specifications. The load settlement factor (γ_{SE}) should be selected on a project-specific basis, typically it is taken as γ_{SE} = 1.0.

AASHTO requires that certain permanent loads and transient loads be factored using maximum and minimum load factors, as shown in Table 8-6 and Table 8-7. The concept of using maximum and minimum factored loads in geotechnical engineering can be associated with using these load factors (max. and min.) to achieve a load combination that produces the largest driving force and the smallest resisting force. Criteria for the application of the permanent load factors (γ_p, γ_{EQ}) are presented below:

- Load factors should be selected to produce the largest total factored force effect under investigation.
- Both maximum and minimum extremes should be investigated for each load combination.
- For load combinations where one force effect decreases the effect of another force, the minimum value should be applied to the load that reduces the force effect.
- The load factor that produces the more critical combination of permanent force effects should be selected from Table 8-6.
- If a permanent load increases the stability or load-carrying capacity of a structural component (e.g., load from soil backfill on the heel of a wall), the minimum value for that permanent load must also be investigated.

Table 8-6, Load Factors for Permanent Loads, γ_p

<table>
<tr><th colspan="2" rowspan="2">Type of Load</th><th colspan="2">Load Factor</th></tr>
<tr><th>Maximum</th><th>Minimum</th></tr>
<tr><td colspan="2">DC: Component and Attachment</td><td>1.25</td><td>0.90</td></tr>
<tr><td colspan="2">DC: Strength IV Only</td><td>1.50</td><td>0.90</td></tr>
<tr><td rowspan="3">DD: Downdrag on Deep Foundations</td><td>Driven Piles (α - Tomlinson Method)</td><td>1.40</td><td>0.25</td></tr>
<tr><td>Driven Piles (λ - Method)</td><td>1.05</td><td>0.30</td></tr>
<tr><td>Drilled Shafts (O'Neill & Reese 1999 Method)</td><td>1.25</td><td>0.35</td></tr>
<tr><td colspan="2">DW: Wearing Surface and Utilities</td><td>1.50</td><td>0.65</td></tr>
<tr><td rowspan="3">EH: Horizontal Earth Pressure</td><td>Active</td><td>1.50</td><td>0.90</td></tr>
<tr><td>At-Rest</td><td>1.35</td><td>0.90</td></tr>
<tr><td>Apparent Earth Pressure (AEP) for Anchored Walls</td><td>1.35</td><td>N/A</td></tr>
<tr><td colspan="2">EL: Locked-in Erection Stresses</td><td>1.00</td><td>1.00</td></tr>
<tr><td rowspan="6">EV: Vertical Earth Pressures</td><td>Overall Stability</td><td>1.00</td><td>N/A</td></tr>
<tr><td>Retaining Walls and Abutments</td><td>1.35</td><td>1.00</td></tr>
<tr><td>Rigid Buried Structure</td><td>1.30</td><td>0.90</td></tr>
<tr><td>Rigid Frames</td><td>1.35</td><td>0.90</td></tr>
<tr><td>Flexible Buried Structures other than Metal Box Culvert</td><td>1.95</td><td>0.90</td></tr>
<tr><td>Flexible Metal Box Culvert</td><td>1.50</td><td>0.90</td></tr>
<tr><td colspan="2">ES: Earth Surcharge</td><td>1.50</td><td>0.75</td></tr>
</table>

The load factors for downdrag loads (DD) are specific to the method used to compute the load. Only maximum load factors for permanent loads (γ_p) are applicable for downdrag loads (DD), these represent the uncertainty in accurately estimating downdrag loads on piles. If the downdrag load acts to resist a permanent uplift force effect, the downdrag load should be considered a resistance and an appropriate uplift resistance factor should be applied.

Earthquake load factors (γ_{EQ}) used in Extreme Event I load combinations should be factored using maximum and minimum load factors, as shown in Table 8-7. These factors are provided for guidance in the design of geotechnical structures where the geotechnical engineer is the engineer-of-record. For the design of bridges, hydraulic structures, and other road structures the SCDOT *Bridge Design Manual* and AASHTO specifications shall be used.

Table 8-7, Load Factors for Earthquake Loads, γ_{EQ}

Type of Load	**Load Factor**	
	Maximum	**Minimum**
LL: Live Load	0.50	0.00
IM: Impact	---	---
CE: Vehicular Centrifugal Force	---	---
BR: Vehicular Breaking Force	---	---
PL: Pedestrian Live Load	0.50	0.00
LS: Live Load Surcharge	0.50	0.00

Table 8-8, Uniform Surcharge Pressures

Material Description		**Uniform Pressure (psf)**
PL: Pedestrian Live Load	Sidewalk widths 2.0 ft or wider	75
	Bridge walkways or bicycle pathways	85
LS[1] : Live load uniform surcharge at bridge abutments perpendicular to traffic Where H_{abut} = Abutment Height	$H_{abut} \leq 5$ ft.	500
	5 ft. $< H_{abut} \leq 20$ ft.	375
	$H_{abut} \geq 20$ ft.	250
LS[1, 2] : Live Load Surcharge on Retaining Walls Parallel To Traffic Where H_{wall} = Wall Height and distance from back of wall = 0.0 ft.	$H_{wall} \leq 5$ ft.	625
	5 ft. $< H_{wall} \leq 20$ ft.	440
	$H_{wall} \geq 20$ ft.	250
LS[1, 2] : Live Load Surcharge on Retaining Walls Parallel To Traffic Where H_{wall} = Wall Height and distance from back of wall $\geq$ 1.0 ft	$H_{wall} \leq 5$ ft.	250
	5 ft. $< H_{wall} \leq 20$ ft.	250
	$H_{wall} \geq 20$ ft.	250
LS[1] : Live Load Surcharge on embankments		250

(1) Uniform Pressure equal to $\gamma_s h_{eq}$ as per AASHTO specifications distributed over the traffic lanes. Where the unit weight of the soil, γ_s, is taken as 125 pcf and the surcharge equivalent height is h_{eq}.
(2) Traffic lanes shall be assumed to extend up to the location of a physical barrier such as a guardrail. If no guardrail or other type of barrier exists, traffic shall be assumed to extend to the back of the wall.

Typical transient loads used to design geotechnical structures for pedestrian live loads (PL), and live load surcharge (LS) shall be computed using the values indicated in Table 8-8. When traffic live loads (LL) are necessary, the AASHTO specifications shall be used.

Dead loads computed for components (DC), wearing surfaces and utilities (DW), and vertical earth pressures (EV) shall be computed using the unit weights of the materials. In the absence of specific unit weights of materials, the values indicated in Table 8-9 should be used.

Table 8-9, Unit Weights of Common Materials

Material Description		Unit Weight (pcf)
Bituminous (AC) Wearing Surfaces		140
Steel		490
Wood	Hard	60
	Soft	50
Unreinforced Concrete[1]	Lightweight	110
	Sand-Lightweight	120
	Normal Weight ($f'_c \leq 5.0$ ksi)	145
	Normal Weight (5.0 ksi < $f'_c \leq 15.0$ ksi) (f'_c - ksi)	140 + f'_c
Soils	Compacted Soils	120
	Very Loose to Loose Sand	100
	Medium to Dense Sand	125
	Dense to Very Dense Sand	130
	Very Soft to Soft Clay	110
	Medium Clay	118
	Stiff to Very Stiff Clay	125
Rock	Rolled Gravel or ballast	140
	Crushed Stone	95
	Gravel	100
	Weathered Rock (PWR)	155
	Basement Metamorphic or Igneous Rock	165
Water	Fresh	62.4
	Salt	64.0

[1] For reinforced concrete, add 5 pcf

8.8 LOAD COMBINATIONS AND FACTORS FOR CONSTRUCTION LOADS

In the design of geotechnical structures the geotechnical engineer must take into consideration potential construction loadings and sequence of construction into the design of geotechnical structures. When a construction method is specified, such as stage construction, and specialty ground improvement (wick drains, surcharges, geosynthetic reinforcement, stone columns, etc.), or when temporary structures such as temporary MSE walls, sheet piling, etc. are designed, the Strength I limit state shall be used with the following modifications to the load factors. The maximum permanent load factor (γ_P) for permanent loads DC and DW shall be at least 1.25 and the maximum load factor for transient loads LL, PL, and LS shall be at least 1.30. Construction plans and specifications of construction methods and temporary construction structures must include construction limitations and sequence of construction used in developing the design.

8.9 OPERATIONAL CLASSIFICATION

Operational classifications have been developed for standard bridges and typical roadway structures. Standard bridges are those bridges whose design is governed by the *Bridge Design Manual*. These classifications have been developed specifically for the South Carolina transportation system. The operational classifications serve to assist in providing guidance as to the operational requirements of the structure being designed. Resistance factors and performance limits in Chapters 9 and 10, respectively, have been established for the various structures based on the operational classification. This is particularly evident when evaluating earthquake engineering analyses/designs. In some cases the degree of analysis or design requirements has been related to the operational classification of the structure. Bridges in the South Carolina transportation system can be classified based on the Bridge Operational Classification (OC) presented in Section 8.9.1 of this Manual. Roadway embankments, retaining structures, and other miscellaneous structures located along the roadways can be classified based on the Roadway Structure Operational Classification (ROC) presented in Section 8.9.2 of this Manual.

8.9.1 Bridge Operational Classification (IC)

The Bridge Operational Classification (OC) presented in Table 8-10 is the same as that used in the SCDOT *Seismic Design Specifications for Highway Bridges*.

Table 8-10, Bridge Operational Classification (OC)

Bridge Operational Classification (OC)	Description
I	These are standard bridges that are located on the Interstate system and along the following roads: ▪ US 17 ▪ US 378 from SC 441 east to I-95 ▪ I-20 Spur from I-95 east to US 76 ▪ US 76 from I-20 Spur east to North Carolina Additional bridges that fall in this category are those structures that meet any of the following criteria: ▪ Structures that do not have detours ▪ Structures with detours greater than 25 miles ▪ Structures with a design life greater than 75 years
II	All bridges that do not have a bridge OC = I and meet any of the following criteria: ▪ A projected (20 years) ADT $\geq$ 500 ▪ A projected (20 years) ADT < 500, with a bridge length of 180 feet or longer or individual span lengths of 60 feet or longer
III	All bridges that do not have a bridge OC = I or II classification.

8.9.2 Roadway Structure Operational Classification (ROC)

The Roadway Structure Operational Classification (ROC) was developed specifically for the Geotechnical Manual to assist in the design of roadway embankments and structures located along the highways. The classification of roadway structures is directly related to the Bridge Operational Classification (OC) by associating proximity to bridges and their respective classification.

Table 8-11, Roadway Structure Operational Classification (ROC)

Roadway Structure Operational Classification (ROC)	Description
I	Roadway embankments or structures located within 150 feet of a bridge with OC = I. Rigid walls with heights greater than 15 feet. Flexible walls with heights greater than 50 feet.
II	Roadway embankments or structures located within 150 feet of a bridge with OC = II.
III	Roadway embankments or structures (retaining walls, etc.) located within 150 feet of a bridge with OC=III or located more than 150 feet from the bridge regardless of the bridge classification.

8.10 LRFD GEOTECHNICAL DESIGN AND ANALYSIS

The limit state that is selected for geotechnical engineering analyses/designs is dependent on the performance limit state and the probability of the loading condition. Guidance in selecting limit states for geotechnical analyses of Bridge Foundations, Earth Retaining Structures, and Embankments are provided in the following subsections.

8.10.1 Bridge Foundations

The design of foundations supporting bridge piers or abutments should consider all limit state loading conditions applicable. Strength limit states are used to evaluate a condition of total or partial collapse. The strength limit state is typically evaluated in terms of shear or bending stress failure.

The Extreme Event I limit state is used to evaluate seismic loadings and its effect on the bridge. The Extreme Event II limit state is used for the evaluation of vessel impact or vehicle impact on the bridge structure. The Extreme Event I limit state may control the design of foundations in seismically active areas. The Extreme Event II limit state may control the design of foundations of piers that may be exposed to vehicle or vessel impacts.

The service limit state is typically evaluated in terms of excessive deformation in the forms of settlement, lateral displacement, or rotation. The Service II and Service III limit states are used to evaluate specific critical structural components and are not generally applicable to foundation design. With respect to deformation, (i.e., horizontal deflection or settlement), the Service I limit state or the Extreme Event limit states will control the design. Performance limits and corresponding limit states for design of shallow foundations and deep foundations are provided in Tables 8-12 and 8-13, respectively.

Bridge foundation design shall take into account the change in foundation condition resulting from scour analyses. The design flood scour (100-year event) shall be used for the strength and service limit states. The scour resulting from a check flood (500-year event) and from hurricanes shall be used for the Extreme Event limit states.

Table 8-12, Shallow Foundation Limit States

Performance Limit	Limit States		
	Strength	Service	Extreme Event
Soil Bearing Resistance	√		√
Sliding Frictional Resistance	√		√
Sliding Passive Resistance	√		√
Structural Capacity	√		√
Lateral Displacement		√	√
Vertical Settlement		√	√

Table 8-13, Deep Foundation Limit States

Performance Limit	Limit States		
	Strength	Service	Extreme Event
Axial Compression Load	√		√
Axial Uplift Load	√		√
Structural Capacity	√		√
Lateral Displacements		√	√
Settlement		√	√

8.10.2 Embankments

The predominant loads influencing the stability of an embankment are dead weight, earth pressure, and live load surcharge. The Strength I limit state load combinations will therefore control the design soil bearing resistance and stability at the Strength limit state. The Service I limit state and the Extreme Event limit states will control the deformation and overall stability of the embankment design. When evaluating the embankment with respect to seismic loads, Extreme Event I limit state is used. The Extreme Event I limit state may control the design in seismically active areas. Performance limits and corresponding limit state for design of embankments are provided in Table 8-14.

Table 8-14, Embankment Limit States

Performance Limit	Limit States		
	Strength	**Service**	**Extreme Event**
Soil Bearing Resistance	√		√
Lateral Spread	√		√
Lateral Squeeze	√		√
Lateral Displacements		√	√
Vertical Settlement		√	√
Overall Stability		√	√

8.10.3 Earth Retaining Structures

The predominant loads influencing the stability of earth retaining structures are dead weight, earth pressure, and live load surcharge. The Strength I and IV limit state load combinations have the largest dead, earth and live load factors and therefore control the design at the Strength limit state. The Strength limit state is evaluated for bearing, sliding, and overturning. The Service I limit state and the Extreme Event limit states will control the deformation performance limits for retaining walls. When evaluating the earth retaining structures with respect to seismic loads, the Extreme Event I limit state is used. The Extreme Event I limit state may control the design in seismically active areas. Performance limits and corresponding limit states for design of earth retaining structures are provided in Table 8-15.

Table 8-15, Earth Retaining Structures Limit States

Performance Limit	Limit States		
	Strength	**Service**	**Extreme Event**
Soil Bearing Resistance	√		√
Sliding Frictional Resistance	√		√
Sliding Passive Resistance	√		√
Structural Capacity	√		√
Lateral Load Analysis (Lateral Displacements)		√	√
Settlement		√	√
Overall Stability		√	√

8.11 REFERENCES

The geotechnical information contained in this Manual must be used in conjunction with the SCDOT *Seismic Design Specifications for Highway Bridges*, SCDOT *Bridge Design Manual*, and AASHTO LRFD Bridge Design Specifications with precedence in order indicated. The geotechnical manual will take precedence over all references with respect to geotechnical engineering design.

AASHTO LRFD Bridge Design Specifications, U.S. Customary Units, 4th Edition, (2007), American Association of State Highway and Transportation Officials.

FHWA-NHI-05-094, (2005). "LRFD for Highway Bridge Substructures and Earth Retaining Structures,", National Highway Institute (NHI), NHI Course No. 130082A, Reference Manual, June 2005.

NCHRP Project 20-7/31, (1986). "Development of Comprehensive Bridge Specifications and Commentary," National Cooperative Highway Research Program (NCHRP)**,** August 1986.

SCDOT *Bridge Design Manual* (2006), South Carolina Department of Transportation, http://www.scdot.org/doing/bridge/06design_manual.shtml

SCDOT *Seismic Design Specifications for Highway Bridges* (2008), South Carolina Department of Transportation, http://www.scdot.org/doing/bridge/bridgeseismic.shtml

Chapter 9

GEOTECHNICAL RESISTANCE FACTORS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Table of Contents

List of Tables

CHAPTER 9

GEOTECHNICAL RESISTANCE FACTORS

9.1 INTRODUCTION

As described in Chapter 8, Resistance Factors (φ) are used in LRFD design to account for the variability associated with the resistance side of the basic LRFD Equation.

$$\boldsymbol{Q \leq \varphi R_n = R_r} \qquad \text{Equation 9-1}$$

Where,

Q	=	Factored Load
R_r	=	Factored Resistance
R_n	=	Nominal Resistance (i.e. ultimate capacity)
φ	=	Resistance Factor

AASHTO and FHWA have conducted studies to develop geotechnical Resistance Factors (φ) based on reliability theory that account for the uncertainties presented below:

- Accuracy of Prediction Models (Design Methodology)
- Site Characterization
- Reliability of material property measurements
- Material properties relative to location, direction, and time
- Material Resistance
- Sufficiency and applicability of sampling
- Soil Behavior
- Construction Effects on Designs

When insufficient statistical data was available, the studies performed a back-analysis of the geotechnical designs to obtain a resistance factor that maintains the current level of reliability that is inferred by the ASD design methodology using the appropriate Factors of Safety.

The LRFD geotechnical design philosophy and load factors for geotechnical engineering are provided in Chapter 8. The Performance Limits for the Service and Extreme Event limit states are provided in Chapter 10. The design methodology used in the application of the design criteria (load factors, resistance factors, and performance limits) is based on AASHTO design methodology with modifications/deviations as indicated in the following Chapters of this Manual:

- Chapter 12 – Earthquake Engineering
- Chapter 13 – Geotechnical Seismic Hazards
- Chapter 14 – Geotechnical Seismic Design
- Chapter 15 – Shallow Foundations
- Chapter 16 – Deep Foundations
- Chapter 17 – Stability and Settlement Analysis and Design
- Chapter 18 – Earth Retaining Structures
- Chapter 19 – Ground Improvement
- Chapter 20 – Geosynthetic Design
- Appendix C – MSE Walls

9.2 SOIL PROPERTIES

The geotechnical Resistance Factors (φ) provided in this Chapter are only appropriate when soil material properties are based on sampling/testing frequency, and testing methods as defined in this Manual. Geotechnical designs and/or analyses should be performed after establishing a "site" based on the site variability with respect to the soil properties that most affect the design or geotechnical analysis. A site variability of "Medium" or lower should be selected based on the requirements of Chapter 7.

Engineering judgment is important in the selection of soil properties but must be used judiciously in a manner that is consistent with the method used to develop the resistance factors and should not be used as a method to account for insufficient geotechnical information due to an inadequate subsurface investigation. As indicated above, the AASHTO resistance factors were developed by either reliability theory or by ASD back-calculation. LRFD resistance factors that were based on reliability theory were developed based on using "average" soil shear properties for each identified geologic unit. LRFD resistance factors that were developed based on a back-analysis of ASD design methodology should use the same method of selecting soil properties (lower bound, average, etc.) as previously used in ASD design. For further information into how the resistance factors were developed the AASHTO LRFD specifications and supporting reference documents should be consulted.

When sufficient subsurface information is available, soil properties should be rationally selected and substantiated by the use of statistical analyses of the geotechnical data. To arbitrarily select conservative soil properties may invalidate the assumptions made in the development of LRFD resistance factors by accounting for uncertainties multiple times; therefore, producing geotechnical designs that are more conservative and consequently have higher costs than the ASD design methodology previously used. When limited amount of subsurface information is available or the subsurface information is highly variable, it may not be possible to select an average soil property for design and a conservative selection of soil properties may be required so as to reduce the risk of poor performance of the structure being designed. Satisfactory performance of the structure outweighs any cost savings that may result from the use of less conservative soil properties.

9.3 RESISTANCE FACTORS FOR LRFD GEOTECHNICAL DESIGN

The geotechnical Resistance Factors (φ) that are provided are distinguished by type of geotechnical structure being designed as listed below.

- Deep Foundations
- Shallow Foundations
- Earth Retaining Structures
- Embankments
- Reinforced Earth Internal Stability

Resistance factors for the determination of liquefaction induced geotechnical earthquake hazards are also provided.

As indicated in Chapter 8, the Fatigue limit state is the only limit state that is not used in geotechnical analyses or designs. Geotechnical resistance factors are provided for the following limit state load combinations:

- Strength – This includes Strength I, II, III, IV, and V.
- Service – This includes Service I
- Extreme Event – This includes Extreme Event I (Seismic loadings) and Extreme Event II (Collision loadings)

Resistance factors are provided based on the type of analysis being performed and the method of determination. When resistance factors are not applicable to the limit state the term "N/A" has been used in the resistance factor tables included in this Chapter. The method of determination shall either be based on the method of construction control or the analytical method used in the design. For details of the analytical methods used in the design see the appropriate chapters in this Manual.

Some analytical methods have not been calibrated for LRFD design methodology. Geotechnical analyses that have not been calibrated include, global stability analyses (static and seismic), and liquefaction induced geotechnical earthquake hazards. For these analyses a load factor (γ) of unity (1.0) should be used. The resistance factors (φ) provided for these analyses is the inverse of the Factor of Safety (1/FS) and consequently have the same margin of safety as previously used in ASD designs. For global stability, Equation 9-1 can be written as indicated below.

$$\frac{\boldsymbol{R_n}}{\boldsymbol{Q}} = \frac{\textbf{Resisting Force}}{\textbf{Driving Force}} = \boldsymbol{FS} \geq \frac{1}{\varphi} \qquad \text{Equation 9-2}$$

Where,

R_n = Nominal Resistance (i.e. ultimate capacity)
Q = Factored Load (With load factor, γ = 1.0)
FS = Factor of Safety
φ = Resistance Factor

The geotechnical Resistance Factors (φ) provided in this Chapter have been selected by the SCDOT based on the standard-of-practice that is presented in this Manual, South Carolina geology, and local experience. Although statistical data combined with calibration have not been used to select regionally specific geotechnical resistance factors, the resistance factors presented in AASHTO and FHWA publications have been adjusted based on substantial successful experience to justify these values. The AASHTO LRFD specifications should be consulted for any geotechnical resistance factors not provided in this Chapter. The PCS/GDS shall review the AASHTO LRFD geotechnical resistance factors that are not included in this Manual prior to approval.

9.4 SHALLOW FOUNDATIONS

Geotechnical Resistance Factors (φ) for shallow foundations have been modified slightly from those specified in the AASHTO LRFD specifications by varying resistance factors based on the structure operational classification (OC or ROC). Resistance factors for shallow foundations are shown in Table 9-1. Resistance factors for bearing resistance are specified for soil and rock. Resistance factors for sliding are based on the materials at the sliding interface.

Table 9-1, Resistance Factors for Shallow Foundations

Performance Limit		Limit States		
		Strength	**Service**	**Extreme Event**
Soil Bearing Resistance (Soil)	OC= I, II, III; ROC = I	0.40	N/A	0.60
	ROC = II or III	0.45		0.65
Soil Bearing Resistance (Rock)	OC= I, II, III; ROC = I	0.40	N/A	0.60
	ROC = II or III	0.45		0.65
Sliding Frictional Resistance (Cast-in-place Concrete on Sand)	OC= I, II, III; ROC = I	0.70	N/A	0.90
	ROC = II or III	0.80		0.95
Sliding Frictional Resistance (Cast-in-place Concrete on Clay)	OC= I, II, III; ROC = I	0.75	N/A	0.90
	ROC = II or III	0.85		0.95
Sliding Frictional Resistance (Precast Concrete on Sand)	OC= I, II, III; ROC = I	0.80	N/A	0.95
	ROC = II or III	0.90		1.00
Sliding Soil on Soil	OC= I, II, III; ROC = I	0.80	N/A	0.70
	ROC = II or III	0.90		0.80
Sliding Passive Resistance (Soil)	OC= I, II, III; ROC = I	0.40	N/A	0.55
	ROC = II or III	0.50		0.65
Lateral Displacement		N/A	1.00	1.00
Vertical Settlement		N/A	1.00	1.00

9.5 DEEP FOUNDATIONS

The design of deep foundations requires that foundations supporting bridge piers or abutments consider all limit state loading conditions applicable to the structure being designed. SCDOT has deviated in its application of LRFD design of deep foundations as presented in the AASHTO LRFD specifications. The deviations are a result of current design and construction practice, design policies, and experience obtained evaluating field load tests of driven piles and drilled shafts. The resistance factors used to determine the nominal resistance for single piles or drilled shafts in axial compression or uplift shall be based on the method of deep foundation load capacity verification during construction. The foundation capacity verification will typically be conducted at Test Pile (non-production piles) locations or at Index Pile (production pile) locations. Foundation capacity verification may be required at any foundation that does not meet foundation installation criteria or whose load carrying capacity is in question. A description of deep foundation load capacity verification methods (wave equation, static load testing, Osterberg cell, dynamic testing, and Statnamic testing) are presented in Chapters 16 and 24. All other resistance factors are based on the design methodology used for deep foundations presented in Chapter 16. The frequency of deep foundation load capacity verification is dependent on the Site Variability as defined in Chapter 7.

The Statnamic load testing method has been included as a method of verifying pile capacity due to its regional popularity and its economic advantages. Statnamic is a relatively new load testing method compared to static load testing or dynamic testing and has yet to be included in the AASHTO specifications. Statnamic load testing is regarded as a load testing method that purportedly falls between a static load test and a dynamic load test. The load applied to the top of the foundation is applied dynamically although at a much slower rate as compared to dynamic testing (PDA). The analysis of the Statnamic load test data requires that the dynamic resistance from the soil be subtracted from the total load applied to obtain the static resistance. Regional experience using Statnamic load testing has shown that dynamic resistance is greater for friction piles/drilled shafts in cohesive soils and consequently the reliability of this method is less for this type of foundation. For friction piles/drilled shafts in cohesionless soils or end-bearing piles/drilled shafts on rock, Intermediate Geomaterial (IGM) or dense sands the dynamic resistance is less and therefore the reliability of the Statnamic load testing method is better when compared to Statnamic load testing of friction piles/drilled shafts in cohesive soils. The method used to separate the dynamic resistance from the static resistance has not been nationally accepted (AASHTO) and the method's reliability has not been independently verified.

SCDOT has conservatively assigned resistance factors for Statnamic load testing based on the limited regional practice. Since cohesive soils tend to produce higher dynamic resistances as compared to cohesionless soils, a lower reliability has been assumed for friction piles/drilled shafts installed in cohesive soils. No increases in resistance factors will be allowed when performing multiple Statnamic tests within a "Site" as indicated in Table 9-4. In order to increase the resistance factors indicated in this Section, a full-scale static load test per "Site" will be required to calibrate the Statnamic load test method of analysis, with the approval of the PCS/GDS. The term "Site" is defined as indicated in Chapter 7.

Another very widely accepted method to verify the axial load capacity of deep foundations is the use of the Osterberg Cell. Since the Osterberg Cell is a type of static load test, the resistance factor for Osterberg Cell load testing method shall be the same as for conventional static load tests indicated in Tables 9-2 and 9-5.

9.5.1 <u>Driven Piles</u>

AASHTO specifications for driven piles differentiate between the predicted nominal axial capacities ($R_{nstatic}$) based on static analyses and the field verified pile capacities (R_n) by applying different geotechnical Resistance Factors (φ) for each of these axial capacities. Upon review of the AASHTO recommended geotechnical Resistance Factors (φ_{stat}) for the static capacity prediction, it was observed that the AASHTO geotechnical Resistance Factors (φ_{stat}) inherently presume a substantial amount of uncertainty in the predicted nominal axial capacity with respect to the field verified pile capacity using either dynamic formula, dynamic analysis, or static load tests. This presumption of greater uncertainty of predicted values vs. field verified values is logical and has merit for a national specification but it does not take into account the regional experience of predicting pile capacities. SCDOT has observed that when using the nominal axial compression pile capacity design methods presented in this Manual that there is rarely a need to extend the pile lengths in the field because the required pile capacity is achieved during pile driving. Driven piles are typically installed in cohesionless soils where pile resistance is most likely underpredicted. The predictive pile capacity method for driven piles installed in the

Cooper Marl has been developed based on pile load tests. It has been observed that the pile capacity methods predict fairly accurately when pile capacity verification is made using pile re-strikes with the Pile Driving Analyzer (PDA). Typically, pile lengths provided in the plans have sufficient length to achieve the required ultimate pile capacity at the end-of-driving or re-strikes when verified by wave equation, dynamic load testing (PDA), or static load tests.

SCDOT has elected to use resistance factors (φ) based on the construction pile capacity verification method required in the plans to predict the nominal axial capacities (static determination of ultimate pile capacity) during design, which is used to select number of piles and pile plan lengths.

Additional considerations that have gone into the selection of SCDOT geotechnical resistance factors are as follows:

- The definition of a “Site” is the same as presented in the AASHTO LRFD specifications with the exception that a “Site” can not have a variability greater than “Medium”. If a “Site” classifies as a “High” variability, the “Site” shall be reduced in size to maintain a variability of “Low” or “Medium.” The Site Variability shall be determined as indicated in Chapter 7.
- Resistance factors are based on a Site Variability of “Low” or “Medium”
- When field load testing is used, a minimum of one test pile is required per “Site” and it is typically placed at the weakest location based on the subsurface soil investigation and design methodology.
- The Contractor’s pile installation plan is reviewed by SCDOT and the pile driving installation equipment is evaluated using the Wave Equation
- Wave Equation Analysis is used to verify the field pile capacity during pile driving. The Wave Equation is calibrated using signal matching (CAPWAP) with the dynamic testing results.
- When load tests are performed, the test pile installation is monitored with the Pile Driving Analyzer (PDA).
- All bridges, regardless of their bridge Operational Classifications (OC), will be designed using the same geotechnical Resistance Factors to maintain the same level of variability.

Load modifiers presented in Chapter 8 are not used to account for the influence of redundancy in geotechnical foundation design. Redundancy in deep foundation design is taken into account by the selection of the geotechnical resistance factor. Non-redundant pile foundations are those pile footings with less than five piles supporting a single column, or less than five piles in a pile bent. Pile footings or pile bents with more than four piles are classified as redundant driven pile foundations.

A resistance factor of 1.0 should be used for soils encountered in scour zones or zones neglected in design when performing pile driveability evaluations or when determining the nominal axial compression capacity to be verified during driving. A resistance factor 10 percent greater than that shown in Table 9-2 can be used for the pile tested, but shall not exceed a resistance factor of 0.80.

Table 9-2, Geotechnical Resistance Factors for Driven Piles

Analysis and Method of Determination	Limit States			
	Strength		Service	Extreme Event
	Redundant	Non-Redundant		
Nominal Resistance Single Pile in Axial Compression with Wave Equation [1]	0.40	0.30	N/A	1.00
Nominal Resistance Single Pile in Axial Compression with Dynamic Testing (PDA) and calibrated Wave Equation [2]	0.65	0.55	N/A	1.00
Nominal Resistance Single Pile in Axial Compression with Static Load Testing. Dynamic Monitoring (PDA) of test pile installation and calibrated Wave Equation [2,3].	See Table 9-4		N/A	1.00
Nominal Resistance Single Pile in Axial Compression with Statnamic Load Testing For Friction Piles. Dynamic Monitoring (PDA) of test pile installation and calibrated Wave Equation [2]	0.65	0.55	N/A	1.00
Nominal Resistance Single Pile in Axial Compression with Statnamic Load Testing For End Bearing Piles in Rock, IGM, or Very Dense Sand. Dynamic Monitoring (PDA) of test pile installation and calibrated Wave Equation [2].	0.70	0.55	N/A	1.00
Pile Group Block Failure (Clay)	0.60	N/A	N/A	1.00
Nominal Resistance Single Pile in Axial Uplift Load with No Verification	0.35	0.25	N/A	0.80
Nominal Resistance Single Pile in Axial Uplift Load with Static Load Testing	0.60	0.50	N/A	0.80
Group Uplift Resistance	0.50	N/A	N/A	N/A
Single or Group Pile Lateral Load Geotechnical Analysis (Lateral Displacements)	N/A	N/A	1.00	1.00
Single or Group Pile Vertical Settlement	N/A	N/A	1.00	1.00
Pile Drivability – Geotechnical Analysis	1.00	1.00	N/A	N/A

(1) Applies only to factored loads less than or equal to 600 kips.
(2) See Table 9-3 for frequency of dynamic testing required.
(3) See Table 9-4 for number of static load testing required.

Dynamic testing is used to control the construction of pile foundations by verifying pile capacity (signal matching required - CAPWAP), calibrating wave equation inspector charts based on signal matching, and monitoring the pile driving hammer performance throughout the project.

In order to use the resistance factors indicated in Table 9-2, a minimum number of Index/Test piles with dynamic testing and signal matching as indicated in Table 9-3 will be required per “Site”. The dynamic testing should be evenly distributed within a “Site”. The test pile locations or bent locations where index piles will be monitored with dynamic testing should be indicated in the plans.

Table 9-3, Test/Index Piles with Dynamic Testing

Number of Driven Piles Located Within a Site	**Number of Test/Index Piles Requiring Dynamic Testing and Signal Matching Analysis**	
	Site Variability	
	Low	**Medium**
≤ 15	3	4
16 – 25	3	5
26 – 50	4	6
51 – 200	4	7
> 200	5	8

All test piles and index piles will require dynamic testing to monitor pile installation. Include additional dynamic testing if restrikes are required for test piles or index piles.

For bridges with 200 or less piles a minimum of 5 additional dynamic tests should be included in the contract to allow for evaluation of poor or highly variable hammer performance or pile restrikes to verify pile capacity throughout the "Site". For bridges with more than 200 piles a minimum 3.0% for "Sites" with "Low" variability or 6.0% for "Sites" with "Medium" variability should be included in the contract to allow for evaluation of poor or highly variable hammer performance or pile restrikes to verify pile capacity throughout the project. The additional dynamic testing of production piles shall be used uniformly throughout the "Site" for QC of the Contractor's pile driving operations.

Table 9-4, Number of Static Load Tests per Site

Number of Static Load Tests per Site	**Resistance Factor (φ)**			
	Low Site Variability		**Medium Site Variability**	
	Redundant	**Non-Redundant**	**Redundant**	**Non-Redundant**
1	0.80	0.65	0.70	0.55
2	0.90	0.70	0.75	0.60
3 or more	0.90	0.70	0.85	0.70

9.5.2 Drilled Shafts

Drilled shaft geotechnical resistance factors (φ) have been provided in Table 9-5. Load resistance factors are provided for Clay, Sand, Rock, and IGM. Statnamic load testing has also been included as indicated in Section 9.5.

Additional considerations that have gone into the selection of SCDOT geotechnical resistance factors are as follows:

- The definition of a "Site" is the same as presented in the AASHTO LRFD specifications with the exception that a "Site" can not have a variability greater than "Medium". If a "Site" classifies as a "High" variability, the "Site" shall be reduced in size to maintain a variability of "Low" or "Medium."
- Resistance factors are based on a site variability of "Low" or "Medium."

- When field load testing is used, a minimum of one test shaft is required per “Site” and it is typically placed at the weakest location based on the subsurface soil investigation and design methodology.

As discussed in Chapter 8, load modifiers will not be used to account for the influence of redundancy in geotechnical foundation design. Redundancy in deep foundations is taken into account by the selection of the geotechnical resistance factor. Non-redundant foundations are those drilled shaft footings with four or less drilled shafts supporting a single column or individual drilled shafts supporting individual columns in a bent. Drilled shaft footings with five or more drilled shafts are classified as redundant drilled shaft foundations. If foundation is a hammerhead (one shaft and one column) reduce the non-redundant resistance factor by 20 percent.

Because drilled shaft capacities can not be verified individually during construction (only drilled shaft installation monitoring), a single resistance factor will be provided for both redundant and non-redundant drilled shafts and no increases in resistance factors will be allowed when performing multiple load tests within a “Site” as indicated in Table 9-4. A resistance factor 10 percent greater than that shown in Table 9-5 can be used for the drilled shaft tested, but shall not exceed a resistance factor of 0.80.

Table 9-5, Resistance Factor for Drilled Shafts

Performance Limit		Limit States			
		Strength		Service	Extreme Event
		Redundant	Non-Redundant (1)		
Nominal Resistance Single Drilled Shaft in Axial Compression in Clay	Side	0.55	0.45	N/A	1.00
	Tip	0.50	0.40	N/A	1.00
Nominal Resistance Single Drilled Shaft in Axial Compression in Sand	Side	0.65	0.55	N/A	1.00
	Tip	0.60	0.50	N/A	1.00
Nominal Resistance Single Drilled Shaft in Axial Compression in IGM	Side	0.70	0.60	N/A	1.00
	Tip	0.65	0.55	N/A	1.00
Nominal Resistance Single Drilled Shaft in Axial Compression in Rock	Side	0.60	0.50	N/A	1.00
	Tip	0.60	0.50	N/A	1.00
Nominal Resistance Single Drilled Shaft in Axial Compression with Static Load Testing		0.70	0.70	N/A	1.00
Nominal Resistance Single Drilled Shaft in Axial Compression with Statnamic Load Testing.		0.65	0.65	N/A	1.00
Nominal Resistance Single Drilled Shaft in Axial Uplift Load (Side Resistance)	Clay	0.45	0.35	N/A	1.00
	Sand	0.55	0.45	N/A	1.00
	IGM	0.55	0.45	N/A	1.00
	Rock	0.50	0.40	N/A	1.00
Nominal Resistance Single Drilled Shaft in Axial Uplift with Static Load Testing		0.60	0.60	N/A	1.00
Drilled Shaft Group Block Failure (Clay)		0.55	N/A	N/A	1.00
Drilled Shaft Group Uplift Resistance		0.45	N/A	N/A	1.00
Single or Group Drilled Shaft Lateral Load Geotechnical Analysis (Structural Capacity)		N/A	N/A	1.00	1.00
Single or Group Drilled Shaft Lateral Load Geotechnical Analysis (Lateral Displacements)		N/A	N/A	1.00	1.00
Single or Group Drilled Shaft Vertical Settlement		N/A	N/A	1.00	1.00

(1) If foundation is a hammerhead (one shaft and one column) reduce the non-redundant resistance factor by 20 percent.

9.6 EARTH RETAINING STRUCTURES

Geotechnical Resistance Factors (φ) for earth retaining structures have been modified slightly from those specified in the AASHTO LRFD specifications by varying resistance factors based on retaining wall system type and the Roadway Structure Operational Classification (ROC). Resistance factors are provided for external stability of the structure with respect to bearing, sliding, and passive resistance. Resistance factors for bearing resistance are specified for soil and rock. Resistance factors for sliding are based on the materials at the sliding interface. For resistance factors due to internal stability of Mechanically Stabilized Earth (MSE) walls see Section 9.8. Resistance factors for Rigid Gravity Retaining Walls are provided in Table 9-6, Flexible Gravity Retaining Walls are provided in Table 9-7, and Cantilever Retaining Walls with or without anchors are provided in Table 9-8.

Rigid Gravity Retaining Walls include cast-in-place concrete walls and brick wall standards typically used in roadway projects. Flexible gravity retaining wall systems include bin walls, panel and block face MSE walls. Cantilever walls include sheet pile walls and soldier pile walls.

Table 9-6, Resistance Factors for Rigid Gravity Retaining Walls

Performance Limit		**Limit States**		
		Strength	**Service**	**Extreme Event**
Soil Bearing Resistance (Soil)	ROC = I, II	0.45	N/A	0.60
	ROC = III	0.45	N/A	0.60
Soil Bearing Resistance (Rock)		0.45	N/A	0.60
Sliding Frictional Resistance (Cast-in-place Concrete on Sand)	ROC = I, II	0.70	N/A	0.90
	ROC = III	0.80		0.95
Sliding Frictional Resistance (Cast-in-place Concrete on Clay)	ROC = I, II	0.75	N/A	0.90
	ROC = III	0.85		0.95
Sliding Frictional Resistance (Precast Concrete on Sand)	ROC = I, II	0.80	N/A	0.95
	ROC = III	0.90		1.00
Sliding Soil on Soil	ROC = I, II	0.80	N/A	0.70
	ROC = III	0.90		0.80
Lateral Displacement		N/A	1.00	1.00
Vertical Settlement		N/A	1.00	1.00
Global Stability Fill Walls	ROC= I, II	N/A	0.65	0.90 (1)
	ROC = III		0.75	1.00 (1)
Global Stability Cut Walls	ROC= I, II	N/A	0.60	0.90 (1)
	ROC = III		0.70	1.00 (1)

(1) Global stability analyses for Extreme Event I limit state that have resistance factors greater than specified require a displacement analysis to determine if it meets the performance limits presented in Chapter 10.

Table 9-7, Resistance Factors for Flexible Retaining Walls

Performance Limit		Limit States		
		Strength	Service	Extreme Event
Soil Bearing Resistance (Soil)		0.55	N/A	0.70
Soil Bearing Resistance (Rock)		0.55	N/A	0.70
Sliding Frictional Resistance (Soil on Soil)		0.90	N/A	0.95
Lateral Displacement		N/A	1.00	1.00
Vertical Settlement		N/A	1.00	1.00
Global Stability Fill Walls	ROC= I, II	N/A	0.65	0.90 (1)
	ROC = III		0.75	1.00 (1)
Global Stability Cut Walls	ROC= I, II	N/A	0.60	0.90 (1)
	ROC = III		0.70	1.00 (1)

(1) Global stability analyses for Extreme Event I limit state that have resistance factors greater than specified require a displacement analysis to determine if it meets the performance limits presented in Chapter 10.

Table 9-8, Resistance Factors for Cantilever Retaining Walls

Performance Limit		Limit States		
		Strength	Service	Extreme Event
Axial Compressive Resistance of Vertical Elements		Section 9.4 Applies		
Passive Resistance of Vertical Element		0.75	N/A	0.85
Flexural Capacity of Vertical Element		0.90	N/A	0.90
Tensile Resistance of Anchor (1)	Mild Steel (ASTM 615)	N/A	0.90 (1)	0.90 (1)
	High Strength Steel (ASTM A 722)		0.80 (1)	0.80 (1)
Pullout Resistance of Anchors (2)	Sand and Silts	N/A	0.65 (2)	0.90 (2)
	Clay		0.70 (2)	1.00 (2)
	Rock		0.50 (2)	1.00 (2)
Anchor Pullout Resistance Test (3) (With proof test of every production anchor)		N/A	1.00 (3)	1.00 (3)
Lateral Displacement		N/A	1.00	1.00
Vertical Settlement		N/A	1.00	1.00

(1) Apply to maximum proof test load for the anchor. For mild steel apply resistance factor to F_y. For high-strength steel apply the resistance factor to guaranteed ultimate tensile strength.

(2) Apply to presumptive ultimate unit bond stresses for preliminary design only. See AASHTO LRFD (C11.9.4.2) specifications for additional information.

(3) Apply where proof tests are conducted on every production anchor to load of 1.0 or greater times the factored load on the anchor.

9.7 EMBANKMENTS

Geotechnical Resistance Factors (φ) for embankments have been modified slightly from those specified in the AASHTO LRFD specifications by varying resistance factors based on the Roadway Structure Operational Classification (ROC). Resistance factors for embankments (fill) sections and cut-sections are shown in Table 9-9.

Table 9-9, Resistance Factors for Embankments (Fill / Cut Section)

Performance Limit		Limit States		
		Strength	Service	Extreme Event
Embankment Soil Bearing Resistance (Soil)		0.55	N/A	0.65
Embankment Soil Bearing Resistance (Rock)		0.55	N/A	0.65
Embankment Sliding Frictional Resistance		0.90	N/A	0.95
Lateral Displacement		N/A	1.00	1.00
Vertical Settlement		N/A	1.00	1.00
Global Stability Embankment (Fill)	ROC= I, II	N/A	0.65	0.90 (1)
	ROC = III		0.75	1.00 (1)
Global Stability Cut Section	ROC= I, II	N/A	0.60	0.90 (1)
	ROC = III		0.70	1.00 (1)

(1) Global stability analyses for Extreme Event I limit state that have resistance factors greater than specified require a displacement analysis to determine if it meets the performance limits presented in Chapter 10.

9.8 REINFORCED SOIL (INTERNAL STABILITY)

Geotechnical Resistance Factors (φ) for analysis of internal stability of reinforced soils are based on AASHTO LRFD specifications. Resistance factors for internal stability of reinforced soils are shown in Table 9-10. Resistance factors may be used in reinforced soil slopes or MSE walls. The external stability of MSE walls shall be governed by the resistance factors provided for flexible walls in Table 9-7. The external stability of Reinforced Steepend Slopes (RSS) shall be governed by the resistance factors provided for embankments in Table 9-9.

Table 9-10, Resistance Factors for Reinforced Soils

Performance Limit		Limit States		
		Strength	Service	Extreme Event
Tensile Resistance of Metallic Reinforcement and Connectors (1)	Strip Reinforcement	0.75	N/A	1.00
	Grid Reinforcement (2)	0.65		0.85
Tensile Resistance of Geosynthetic Reinforcement And Connectors		0.90	N/A	1.20
Pullout Resistance of Tensile Reinforcement		0.90	N/A	1.00
Sliding at Soil Reinforcement Interface		0.80	N/A	1.00

(1) Apply to gross cross-section less sacrificial area. For sections with holes, reduce the gross area and apply to net section less sacrificial area.

(2) Applies to grid reinforcements connected to a rigid facing element (concrete panel or block). For grid reinforcements connected to a flexible facing mat or which are continuous with the facing mat, use the resistance factor for strip reinforcements.

9.9 LIQUEFACTION INDUCED GEOTECHNICAL EARTHQUAKE HAZARDS

Geotechnical Resistance Factors (φ) for shear strength loss (SSL) and SSL-induced geotechnical seismic hazards are provided in Table 9-11. Resistance factors for other earthquake hazards that are not liquefaction induced (i.e. seismic slope stability, lateral foundation displacements, downdrag on deep foundations, etc.) are addressed under the Extreme Event limit state for each specific structure. These resistance factors apply only to the Extreme Event I limit state.

Table 9-11, Resistance Factors for Soil Shear Strength Loss Induced Seismic Hazards

Earthquake Hazard Description	Resistance Factor Symbol φ	Design Earthquake	
		FEE	SEE
Sand-Like Soil Shear Strength Loss (Liquefaction) (Triggering)	$\varphi_{SL\text{-}Sand}$	0.85	0.90
Sand-Like Soil No Shear Strength Loss (No Liquefaction)	$\varphi_{NSL\text{-}Sand}$	0.70	0.75
Clay-Like Soil Shear Strength Loss (Triggering)	$\varphi_{SL\text{-}Clay}$	0.85	0.90
Flow Failure (Triggering)	φ_{Flow}	0.90	0.95
Lateral Spread (Triggering)	φ_{Spread}	0.90	0.95
Site R/W Seismic Instability (Triggering)	$\varphi_{EQ\text{-}Stability}$	0.90	0.95

9.10 REFERENCES

The geotechnical information contained in this Manual must be used in conjunction with the SCDOT Seismic Design Specifications for Highway Bridges, SCDOT Bridge Design Manual, and AASHTO LRFD Bridge Design Specifications. The geotechnical manual will take precedence over all references with respect to geotechnical engineering design.

AASHTO LRFD Bridge Design Specifications, U.S. Customary Units, 4th Edition, (2007), American Association of State Highway and Transportation Officials.

SCDOT Bridge Design Manual (2006), South Carolina Department of Transportation, http://www.scdot.org/doing/bridge/06design_manual.shtml

SCDOT Seismic Design Specifications for Highway Bridges (2008), South Carolina Department of Transportation, http://www.scdot.org/doing/bridge/bridgeseismic.shtml

Chapter 10

GEOTECHNICAL PERFORMANCE LIMITS

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Table of Contents

List of Tables

List of Figures

CHAPTER 10

GEOTECHNICAL PERFORMANCE LIMITS

10.1 INTRODUCTION

LRFD incorporates the use of limit states as a condition beyond which a component/member or foundation of a structure ceases to satisfy the provisions for which it was designed. The Service limit states and the Extreme Event limit states have design boundary conditions for structural performance that account for some acceptable measure of structural movement throughout the structure's design life. The performance limits for geotechnical structures such as Embankments, Bridge Foundations, and Earth Retaining Structures (ERS) are presented in this Chapter. Performance limits include the design life of the structure, structural performance under Service loads, and structural performance under Extreme Event Loads. The design life for bridge structures is typically 75 years and for geotechnical structures 100 years. Structures that cannot be replaced without significant expense or that may be subject to structural distress due to environmental conditions (corrosion, biological degradation, etc.) may have a design life that exceeds the typical design life. The structural performance under Service and Extreme Event loads are typically expressed in terms of settlement, settlement rate, differential settlement, vertical displacement, lateral displacements, and rotations.

The LRFD geotechnical design philosophy and load factors for geotechnical engineering are provided in Chapter 8. The resistance factors for the Strength, Service, and Extreme Event limit states are provided in Chapter 9. The design methodology to analyze structure performance shall be in accordance with AASHTO design methodology with modifications/deviations as indicated in the appropriate Chapters of this Manual.

10.2 PERFORMANCE LIMITS FOR LRFD GEOTECHNICAL STRUCTURES

Transportation structures are typically thought of as being rigid and stationary, but in reality they deform throughout their service life due to various physical (loads) and environmental (temperature, degradation, etc.) conditions exerted on the structures. The deformations range from the elastic range where no permanent deformations remain after unloading, to the plastic range where deformations become permanent even after unloading, and finally to rupture where the material is permanently severed. The types of loadings that cause these deformations are discussed in Chapter 8. The deformations experienced by geotechnical structures are typically non-linear, dependent on subsurface site variability, influenced by environmental factors, and highly dependent on soil-structure interaction due to strain compatibility (stiffness) between soil, aggregates (stone, gravel, etc.), soil reinforcements/anchors (steel or geosynthetic), reinforced concrete, steel, etc. Soils are considerably more compressible, have essentially no tensile strength, and have shear strengths that occur at considerably larger displacements. Unlike concrete and steel, soil properties are highly variable. Soils found in-place may vary significantly within short distances both vertically and horizontally because soil composition and properties are based on geologic mechanisms. When soils are engineered through material selection and construction control, soil variability in composition and density can still occur as a result of the non-uniformity of the material stockpile, weather, and construction.

Performance limits presented in this Chapter are the result of the SCDOT first establishing Performance Objectives for typical geotechnical structures such as Embankments, Bridge Foundations, and Earth Retaining Structures. Once the Performance Objectives are established, the Performance Limits for each geotechnical structure were developed to meet the level of functionality defined by the objectives.

Performance Objectives and Performance Limits define the level of functionality of the structure for the limit state loading condition being evaluated. The Performance Objectives and Performance Limits for permanent geotechnical structures in this Chapter are based on:

- Limit State: Service I limit state or Extreme Event I limit state load combinations defined in Chapter 8.
- Geotechnical Structure Importance Classification: Bridge Operational Classification (OC) or Roadway Structure Operational Classification (ROC) as defined in Chapter 8.

The loadings used in these analyses are typically without adjustment for variability in both the load and resistance portion of the analysis. The load and resistance factors generally used in geotechnical analyses are unity (1.0) unless indicated otherwise in Chapters 8 and 9 of this Manual. When load factors greater than unity (1.0) or resistance factors less than unity (1.0) are used, this is typically due to the variability or uncertainty associated with the load or resistance being computed. The design intent is to analyze the most likely behavior of the structure when subjected to typical loadings for each limit state.

The Performance Objectives and Performance Limits for the following geotechnical structures are not provided in this Manual and should be developed on a project specific basis.

- Performance Objectives and Limits for Hydraulic Structures (three-sided culverts, concrete box culverts, etc.) at the Service I and Extreme Event I limit state
- Extreme Event II performance limits for collision loadings
- Temporary geotechnical structures (i.e. structures having a life of less than 5 years)

It is the intent of this Chapter to also provide the framework to develop project specific performance objectives and limits for structures subjected to service loadings and Extreme Event loadings that are not included in this Chapter.

When evaluating the performance of hydraulic structures consideration of adjacent structures such as Embankments (Section 10.7) or Earth Retaining Structures (Section 10.9) should be given since the Performance Objectives and Performance Limits of these geotechnical structures may not be compatible with the requirements for hydraulic structures.

10.2.1 Service Limit State Performance Objectives

The Performance Objective for the Service limit state (SLS) requires that with standard SCDOT maintenance the structure remain fully functional to normal traffic for the design life of the structure. The performance of a structure under Service loads is influenced by many factors that may or may not be within the designer’s control. The following list of considerations will influence the Service performance of the structure over its design life.

Safety The structure must be designed safely so as not to collapse and to control structural damage so as to reduce the risk of loss of life. The reliability of the design to maintain this objective is addressed by designing for the Strength limit state that takes into account the variability of the load and resistance. Structures that are structurally designed for the Strength limit state will have component/members and foundations that are sized for larger loadings than loadings observed at the Service limit state. Having components/members and foundations of a structure first sized for Strength limit state typically improves the performance of the structure by increasing the stiffness of the members. This results in smaller deformations and improved performance.

Accepted design methodologies for evaluating the global stability of a structure at the Strength limit state are not currently available. Currently, global stability is evaluated at the Service limit state using appropriate resistance factors that provide for designs that are the equivalent of Allowable Stress Designs (ASD). This method of evaluating global stability assumes that the driving and resistance forces are maintained in equilibrium within an appropriate safety margin and therefore no displacements occur. The performance limit for the global stability at the Service limit state is that no displacements occur over the life of the structure.

Operational Classification SCDOT has established operational classifications for typical bridges (OC) and roadway structures (ROC) to allow for differentiation between structures of higher and lower operational requirements to the South Carolina transportation infrastructure. The operational classification has three levels I, II, and III where level I is the highest operational classification and level III is the lowest operational classification. The bridge structure operational classification (OC) and the roadway structure operational classification (ROC) are defined in Chapter 8. This classification allows SCDOT to vary the reliability and performance expectations between structures that have relatively high operational requirements such as the Interstate system to those on low volume roads that are typically part of the secondary roadway system.

Design Life This is the anticipated life expectancy of the structure until it will require replacement by a new structure. It is assumed that the structure has periodic inspection and maintenance so as not to reduce the expected Design Life.

Functionality Functionality of a structure requires acceptable performance of the structure in order to be useable by the traveling public. This is accomplished by establishing performance limits (traffic projections, deformation limits, rideability requirements, etc.) for the Design Life of the structure. In order to maintain the required functionality of the structure, periodic maintenance will be required.

Aesthetics The Service limit state requires that the aesthetics of a structure be consistent with the environment where the structure will be placed. The aesthetic requirement of a structure located in an urban setting with high visibility will be different from those aesthetic requirements of a structure located in a rural setting with low visibility by the traveling public. Aesthetics of the structure is also defined by public perception of how safe or visually appealing a structure appears. A structure that is structurally stable but has cracks, excessive deformations in the form of bulges, out-of-plumb, etc. is not aesthetically satisfactory. Satisfying aesthetics objectives requires proper planning (public hearings, timely information, etc.), good construction specifications that specify construction tolerances, finish requirements, proper inspection during construction, and periodic maintenance.

Construction The Service limit state requires the development of plans and construction specifications that are clear and take into account the constructability of the design and any construction monitoring. Construction specifications should include construction tolerances, construction methods, and field performance monitoring of the structure such as settlement monitoring.

Maintenance A Maintenance Plan should be in place that consists of periodic inspections of the structure and communication with designers to evaluate the results of the inspections. The Maintenance Plan should also provide for the development of the appropriate responses required to meet the serviceability requirements of the structure for the remainder of its design life. Design details of the structure should allow for periodic inspection of vital components that would affect the structure's performance.

Risk The selection of the type of structure to be used in the design should consider any associated risk that would affect the performance of the structure. Some factors that increase the risk of unsatisfactory structure performance are presented below:

- *Construction*: Common types of structures are usually associated with less construction risk due to the familiarity of the construction procedures.
- *Structure Selection*: Failure to consider the limitations of the structure type selected in relation to the desired performance may lead to unsatisfactory performance. A common misapplication in construction is the use of cantilever sheetpiling for temporary shoring of deep excavations. The deformations typically exceed acceptable performance for adjacent structures.
- *Design/Construction Methodology*: Misapplication of methodologies in design (i.e. using unaccepted design methods) or construction (i.e. misapplication of ground improvement method).
- *Design Experience*: Insufficient design experience of either the structure design or any ground improvement required can lead to unsatisfactory performance. Insufficient design experience includes untested designs, new design methodologies, and designer's inexperience.

- *Geotechnical Investigation*: A subsurface geotechnical investigation that does not adequately describe the foundation soils can lead to construction delays, "changes in soil/subsurface conditions", redesign of foundations that unfortunately results in contractor claims, increased construction costs, not meeting schedules, litigation, etc. The long-term impacts of an inadequate geotechnical investigation can result in poor long-term performance of the structure that results in higher maintenance costs and in many cases replacement of the structure before it has reached its expected design life.
- *Change in Soil/Subsurface* Conditions: These are unforeseen field conditions that typically cannot be accounted for during design. This situation is also referred to as "Differing Site Conditions." When changes in soil/subsurface conditions occur, they can be addressed during construction with proper communication between Construction and Design personnel. Field conditions that fall into this category are subsurface soil variability, and environmental factors (weather, etc.). Performing an adequate geotechnical subsurface investigation during the design of the structure is the most cost effective method of reducing the risk of having a "change in soil/subsurface conditions" from occurring during construction.

Quantifiable Performance Limits are needed, therefore Design Life and Deformation Limits are the only criteria defined for the Service limit state. Where possible, the factors listed above have been taken into consideration in the development of the performance limits listed for the Service limit state.

10.2.2 Extreme Event Limit State Performance Objectives

The Extreme Event limit states (EE I and EE II) are load combinations that are typically in excess of the Service limit state loadings and in some cases may also be in excess of the Strength limit state. The loadings from these Extreme Events are typically the result of earthquake events or collisions from ships, barges, or vehicles. The Extreme Event limit states have the potential to cause damage to a structure and impact the structure's functionality. Even though Extreme Event limit states typically have a low probability of occurring within the design life of the structure, these limit states loadings must be evaluated because the potential for loss of life and loss of service of the structure can be significant. Because the probability of these events occurring is relatively low, a lower safety margin is used and performance limits are less rigid than those for the Service limit state. The damage resulting from these Extreme Event loading conditions may be significant enough to warrant replacement of the structure, but under no design condition should the structure be allowed to collapse.

The Performance Objectives for the Extreme Event limit state of a structure are defined by selecting an appropriate Service Level and Damage Level for each component/member or foundation element being analyzed. For complex structures such as bridges and earth retaining structures, performance objectives are first given to the overall structure and then component performance objectives are given to the individual component/members or foundation of the structure. Although this approach is somewhat subjective at this time, it allows for a more methodical way of evaluating each component of the structure to meet the overall performance objective of the complete structure.

Service Level refers to the ability to repair the structure (if necessary) and return the structure to a specified level of service within a prescribed amount time. The following Service Level descriptions are used in this Manual to define the Service Level Performance Objectives for the Extreme Event limit states.

Table 10-1, Extreme Event Service Level

Service Level	Description
Immediate	Full access to normal traffic is available immediately following the event.
Maintained	Immediately open to emergency traffic. Short period of closure to the Public with access typically within days of the event.
Recoverable	Limited period of closure to Public with access typically within weeks to months after the event.
Impaired	Extended closure to Public with access typically restored within months to years after the event.

Damage Level implies that there is an acceptable degree of damage that a structure can undergo. Although damage may be allowed to occur, complete collapse of the structure where loss of life may occur is not acceptable. When developing Performance Objectives the reliability of the Extreme Event loadings should be considered with respect to the potential consequences to the overall structure should an individual component/member or foundation reach structural failure. The following Damage Level descriptions are used in this Manual to define the Damage Level Performance Objective for the Extreme Event limit states.

Table 10-2, Extreme Event Damage Levels

Damage Level	Description
Minimal	No collapse, essentially elastic performance (No permanent deformations)
Repairable	No collapse, concrete cracking, spalling of concrete cover, and minor yielding of structural steel will occur. However, the extent of damage should be sufficiently limited such that the structure can be restored essentially to its pre-earthquake condition without replacement of reinforcement or replacement of structural members. Damage can be repaired with a minimum risk of losing functionality.
Significant	Although there is minimum risk of collapse, permanent offsets may occur in elements other than foundations. Damage consisting of concrete cracking, reinforcement yielding, major spalling of concrete, and deformations in minor bridge components may require closure for repair. Partial or complete demolition and replacement may be required in some cases.

The Extreme Event I limit state is a load combination that is associated with a Design Earthquake event. The SCDOT uses the Design Earthquakes listed in Table 10-3. Additional information concerning these design earthquakes can be found in Chapters 11 and 12. The Performance Objectives and seismic design requirements for bridges are provided in the latest edition of the *SCDOT Seismic Design Specifications for Highway Bridges* and in this Manual.

Table 10-3, SCDOT Design Earthquakes

Design Earthquake	Description
Functional Evaluation Earthquake (FEE)	The ground shaking having a 15 percent probability of exceedance in 75 years (15%/75 year). This design earthquake is equal to the 10 percent probability of exceedance in 50 years (10%/50). The FEE PGA and PSA are used for the functional evaluation of transportation infrastructure.
Safety Evaluation Earthquake (SEE)	The ground shaking having a 3 percent probability of exceedance in 75 years (3%/75 year). This design earthquake is equal to the 2 percent probability of exceedance in 50 years (2%/50). The SEE PGA and PSA are used for the safety evaluation of transportation infrastructure.

10.2.3 Performance Limits

The Performance Limits that are specified in this Manual are for new construction and do not apply to retrofitting or maintaining existing structures. Performance Limits have been developed based on SCDOT design and construction standards of practice contained in this Manual, SCDOT *Bridge Design Manual*, SCDOT *Seismic Design Specifications for Highway Bridges*, and in accordance with SCDOT construction specifications. AASHTO and FHWA publications and SCDOT experience have been used as the basis to establish the SCDOT Performance Limits. SCDOT reserves the right to change these Performance Limits based on project specific requirements or as new research or as additional experience becomes available. The Performance Limits specified in this Manual are upper limits based on typical structures used in South Carolina. The designer, with concurrence of the PCS/GDS, may impose more restrictive Performance Objectives and Limits depending on the type of structure and its operational classification. The designer of the structure (engineer-of-record) has the ultimate responsibility to ensure that the Performance Limits provided in this Manual are used judiciously so as not to place in jeopardy the Performance Objectives of the structure being designed. It is the geotechnical engineer's responsibility to present the geotechnical performance findings to the designer and to assist the designer in evaluating geotechnical and structural solutions for maintaining the structure's performance within acceptable limits.

Performance Limits specified in this Chapter are specific to the type of structure being designed. The acceptable deformations specified are based on the structure's intended use as provided in the Service limit Performance Objectives for Embankments (Section 10.7), Bridges Foundations (Section 10.8), and Earth Retaining Structures (Section 10.9). Performance Limits may need to be adjusted for these structures based on any adjacent structures such as hydraulic structures, utilities (water, gas, electricity, phone, etc.), pavements, bridges, retaining walls, signs, homes, buildings, etc. that may be impacted by the deformations that are deemed acceptable for the structures that are addressed in this Manual. For example, settlements that may be acceptable for an embankment may not be acceptable for an existing building within the influence of a roadway embankment. Another example where Performance Limits provided may not be acceptable would be during global instability, where deformations of an embankment may distress adjacent structures such as bridges, side ramps, or other structures beyond the Right-of-Way.

Performance Limits not covered in this Manual will require that the designer, in conjunction with the SCDOT, first establish Performance Objectives for the structure being analyzed. Once the Performance Objectives have been developed, Performance Limits can be established to meet the Performance Objectives.

10.3 DEFORMATIONS

Performance Limits are specified in terms of acceptable vertical and lateral displacements. Displacements can be a result of direct movements such as settlement of an embankment or as a result of rotations such as embankment instability or foundation rotations due to lateral loadings. Vertical displacements that occur in a downward direction (into the ground) are referred to as settlement. Specifying a Maximum Vertical Settlement can help to control total settlements. Damage or poor performance of a structure most often occurs as a result of excessive differential displacements. An example of this would be a bridge with foundations supported by rock and with the approach embankments supported on very compressible soils. The bridge would remain relatively stationary vertically while the approach embankment would settle substantially relative to the bridge. The vertical differential displacements would affect vehicle rideability and add structural loads to the abutment foundations as a result of downdrag on deep foundations. Specifying a Maximum Vertical Differential Settlement would help to control the differential vertical displacements that occur between the bridge abutment and the bridge approach embankment to an acceptable level of performance. There may be situations where vertical displacements act upward, due to heave or differential movements of a structure. This condition may cause part of the structure to move up when other parts of the structure move downward (settlement). The Maximum Vertical Differential Displacement limits also control these upward and downward displacements to an acceptable level of performance.

Lateral displacements (horizontal movements) are identified as occurring in either longitudinal or transverse directions. On bridges and roadways, the longitudinal direction is the same direction as the vehicle travel direction (either travel lane). The transverse direction is the direction that is perpendicular to the vehicle travel direction. Unless otherwise indicated in the performance limit description, the lateral displacements do not have sign convention and may occur in either direction.

10.4 EMBANKMENT DEFORMATIONS

10.4.1 Embankment Terminology and Deformation Notations

Embankment design with respect to global stability and settlements are discussed in Chapter 17. Terminology used to specify geotechnical performance limits for embankments along roadways and at bridge approaches is presented in Table 10-4.

Table 10-4, Embankment Terminology

Terminology	Description
Embankment	An earthen mass structure constructed from select fill material. Fill materials are placed in compacted lifts over competent soil capable of supporting the structure.
Bridge Embankment	The embankment that extends 150 feet longitudinally from the "begin" or "end" of bridge and extends to the toe of the front and side slopes. The approach embankment classification may be extended if there are any stability or settlement issues that would affect the bridge performance or transition between the embankment and the bridge.
Front Slope	The embankment that extends longitudinally beneath the bridge. The front slope begins at the end bent and extends to the existing ground surface. Front slope grades are given in ratios of horizontal distance to vertical height (i.e. 2(H):1(V)).
Side Slopes	The embankment that extends perpendicular to the travel lane and has been graded to meet traffic safety and stability requirements. The side slope begins at the edge of the roadway and extends to the existing ground surface. Side slope grades are given in ratios of horizontal distance to vertical height (i.e. 3(H):1(V)), transverse to the roadway travel direction.
Profile Grade	Roadway plans typically have plan and profile sheets. The profiles are given along a specific location of the pavement surface that is to referred in the plans as the Profile Grade (P.G.) or Finished Grade (F.G.). Often this location is the same as the centerline of the road. There may be multiple profile grades along a divided roadway or intersection for each traffic direction. The location of the roadway alignment in plan view typically coincides with the location of the profile grade.
Alternate Profiles	Alternate profiles are sometimes necessary when evaluating settlements. These profiles are typically parallel the alignment of the roadway at a location that is subject to larger settlements than those at the Profile Grade location.
Cross-Section	A slice or section taken perpendicular to the roadway alignment at a specific location (station) of the road.
Station	Locations along reference base line on the plan or profile that is based on measurements from a reference point (i.e. Sta. 1+00.00 = 100.00 feet).
Global Stability Analysis	An estimation of the balance between the driving force and resisting force within an earthen mass that is seeking to reach equilibrium.
Global Instability	An imbalance of equilibrium of an earthen mass that causes a failure shear surface to occur and consequently the earthen mass deforms.
Failure Surface	An approximation of the most likely shear failure surface that will develop as a result of instability of an earthen mass.
Approach Slab	A reinforced concrete structural slab placed on the embankment to transition from the roadway pavement to the bridge surface at the end bent. Approach slabs are typically 20 feet in length.

Embankment deformation notations are listed in Table 10-5. Embankment deformations where Performance Limits are specified can be categorized as follows:

- Global Instability Deformations (Section 10.4.2)
- Embankment Settlement (Section 10.4.3)
- Embankment/Bridge Transition Settlement (Section 10.4.4)
- Embankment Widening Settlement (Section 10.4.5)

Table 10-5, Embankment Deformation Notations

Notation	Description
δ_V	Vertical Differential Settlement
Δ_V	Vertical Displacement / Settlement
Δ_{VP}	Vertical Settlement at a Profile Grade at a specific Station (cross-section).
Δ_{VA}	Vertical Settlement at end of Approach Slab/Embankment
Δ_{VE}	Vertical Settlement at the End Bent (Abutment).
Δ_{VT}	Vertical Settlement of new embankment widening section at location of maximum settlement.
Δ_{VTS}	Vertical Displacement at the Top of the Slope failure surface
Δ_{VBS}	Vertical Displacement at the Bottom of the Slope failure surface
Δ_L	Lateral Displacement
Δ_{LTS}	Lateral Displacement at the Top of the Slope failure surface
Δ_{LBS}	Lateral Displacement at the Bottom of the Slope failure surface
ΔL	Deformation occurring along the critical failure surface due to slope instability.
L_{Slab}	Longitudinal Length of the approach slab
L_L	Longitudinal distance of area affected by the compressive soils producing embankment settlements.
L_T	Transverse distance that defines the span of maximum differential settlement from the existing embankment (no settlement or minimal settlement) to the location of maximum settlement for the portion of new embankment that has been widened.

10.4.2 Global Instability Deformations

Embankment global instability deformations are not analyzed at the Service limit state since the design methodology for global stability analyses (Chapter 17) requires that the global stability analyses maintain a specified margin of safety (resistance factor, φ) against instability. Deformations only occur when there is an imbalance of equilibrium of the earthen masses. Because performance objectives for the Extreme Event limit state permit an acceptable amount of deformation, global instability and consequent deformation analyses must be made for the Extreme Event limit state. Embankment deformations associated with the Extreme Event I (EE I) limit state (earthquake loadings) include flow slide, lateral spread, seismic instability, and seismic settlement. Deformations associated with flow slides and lateral spread are assumed to exceed performance limits for the EE I limit state and must be mitigated. Because methods of analyzing deformations due to limited lateral spread and seismic instability are provided in Chapter 13, performance limits have been developed that address these types of deformations. Performance Limits for global instability deformations are identified in Table 10-6.

Table 10-6, Global Instability Deformations Performance Limits

Notation	Deformation ID No.	Description
Vertical Displacement, Δ_V	**GI-01**	Maximum Vertical Displacement (Δ_{VTS}) at top of the failure surface.
	GI-02	Maximum Vertical Displacement (Δ_{VBS}) at bottom of the failure surface.
Lateral Displacement, Δ_L	**GI-03**	Maximum Lateral Displacement (Δ_{LTS}) at top of the failure surface.
	GI-04	Maximum Lateral Displacement (Δ_{LBS}) at bottom of the failure surface.

Extreme Event I limit state performance limits for global instability deformations associated with limited lateral spread and seismic slope instability are specified along the shear failure surface that results from the imbalance in equilibrium of the slope. Performance Limits GI-01 and GI-03 are located at the top of the failure surface and GI-02 and GI-04 are located at the bottom of the failure surface.

Global instability deformations can occur at:

- Roadway Embankment Side Slopes as shown in Figures 10-1 and 10-2.
- Bridge Approach Embankments as shown in Figures 10–9 and 10–12.
- Earth Retaining Structures as shown in Figures 10–14 and 10–15.

The evaluation of global instability deformations is very complex and the methods (Chapter 13) that have been developed to evaluate deformations are typically either empirical or are very simplistic models that only provide an approximation of the slope instability deformations. A considerable amount of engineering judgment will be required to evaluate embankment deformations. To simplify this evaluation, it can be assumed that the soil is incompressible and deformations occur equally along the critical failure surface. The embankment deformations at the top of the slope can be roughly estimated by computing the displacement components (Δ_{LTS} and Δ_{VTS}) from the deformation ΔL acting along the critical failure surface. The embankment deformations at the bottom or toe of the slope can be roughly estimated by computing the displacement components (Δ_{LBS} and Δ_{VBS}) from the deformation ΔL as it projects tangentially to the failure surface at the intersection with the original ground surface configuration. Embankment deformations due to global instability for circular and sliding block failure surfaces are shown in Figures 10-1 and 10-2, respectively.

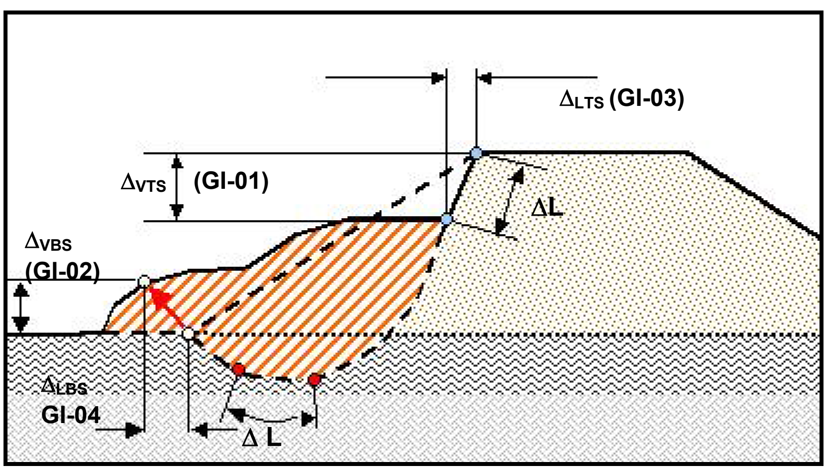


Figure 10-1, Embankment Circular Arc Instability

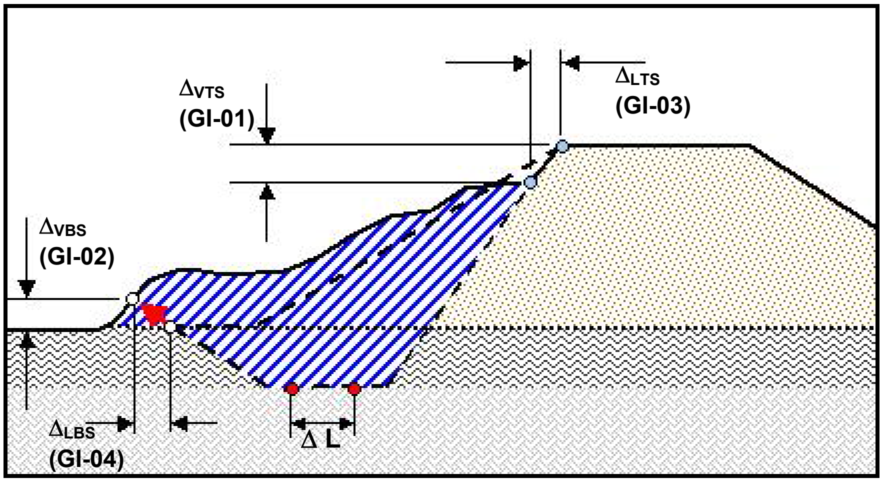


Figure 10-2, Embankment Sliding Block Instability

10.4.3 Embankment Settlement

Embankment vertical settlements are typically due to embankments being constructed over compressible soils that experience soil deformation (elastic compression, primary consolidation, and secondary consolidation) under constant load. Settlement analysis methods are provided in Chapter 17 of this Manual. The vertical settlements that are evaluated under the Service I limit state are as indicated below.

- Maximum Settlement from Elastic Compression + Primary consolidation
- Maximum Settlement Rate from Primary Consolidation + Secondary Consolidation
- Maximum Differential Settlement from Primary Consolidation + Secondary Consolidation

Under the Extreme Event I limit state, performance limits for embankment settlement are specifically those caused by geotechnical seismic hazards that may affect the embankment or subgrade during or after a seismic event. Methods of analyzing geotechnical seismic hazards due to liquefaction of the subgrade or seismic settlement of the embankment and subgrade are discussed in Chapter 13.

Performance limits for embankment settlements are identified in Table 10-7.

Table 10-7, Embankment Settlement Performance Limits

Notation	Deformation ID No.	Description
Vertical Settlement, Δ_V	EV-01	Maximum Settlement from Elastic Compression + Primary consolidation along the profile grade (Δ_{VP}) over the design life of the embankment. The design life begins after the pavement has been placed.
	EV-02	Maximum Settlement Rate from Primary Consolidation + Secondary Consolidation per year after the roadway has been paved.
Vertical Differential Settlement, δ_V	EV-03	Maximum Differential Settlement from Primary Consolidation + Secondary Consolidation occurring longitudinally along the profile grade after the roadway has been paved

The roadway Profile Grade (P.G.) for non-divided highways (highways without medians) is typically located at the center of the roadway as indicated in Figure 10-3. Figure 10-3 is designated as Section A-A that corresponds to an embankment cross-section taken transverse to the travel lane as indicated in Figure 10-5. Embankment settlements are evaluated at the center of embankment sections where the maximum settlements are most likely to occur and consequentially also where the maximum differential settlements occur.

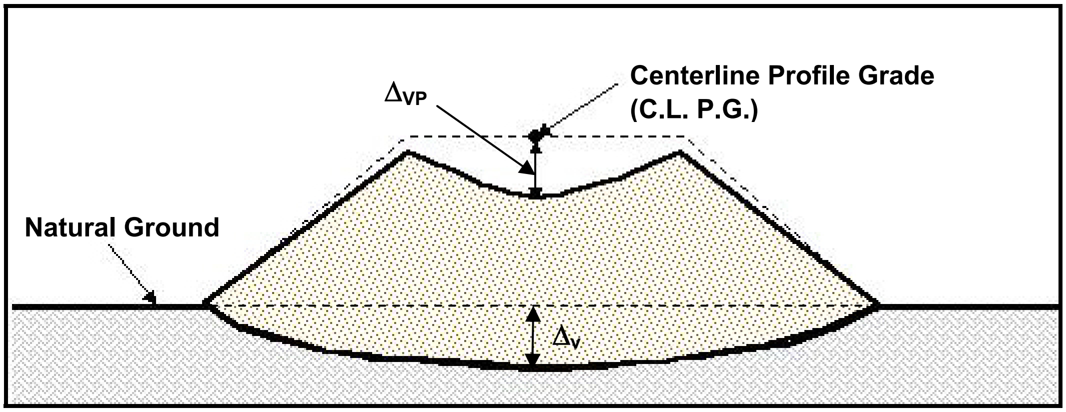


Figure 10-3, Embankment Settlement (Section A–A)

Divided highways may have a Profile Grade (P.G.) elevation for each travel direction as indicated in Figure 10-4. Figure 10-4 is designated as Section A-A that corresponds to an embankment cross-section taken transverse to the travel lane as indicated in Figure 10-5. To differentiate the divided profile grades the color Blue was used to designate the roadway on the left and the color Red was used to designate the roadway on the right. Divided highways should be evaluated separately for each P.G. Settlement analyses must take into account the total embankment cross-section and the construction sequencing.

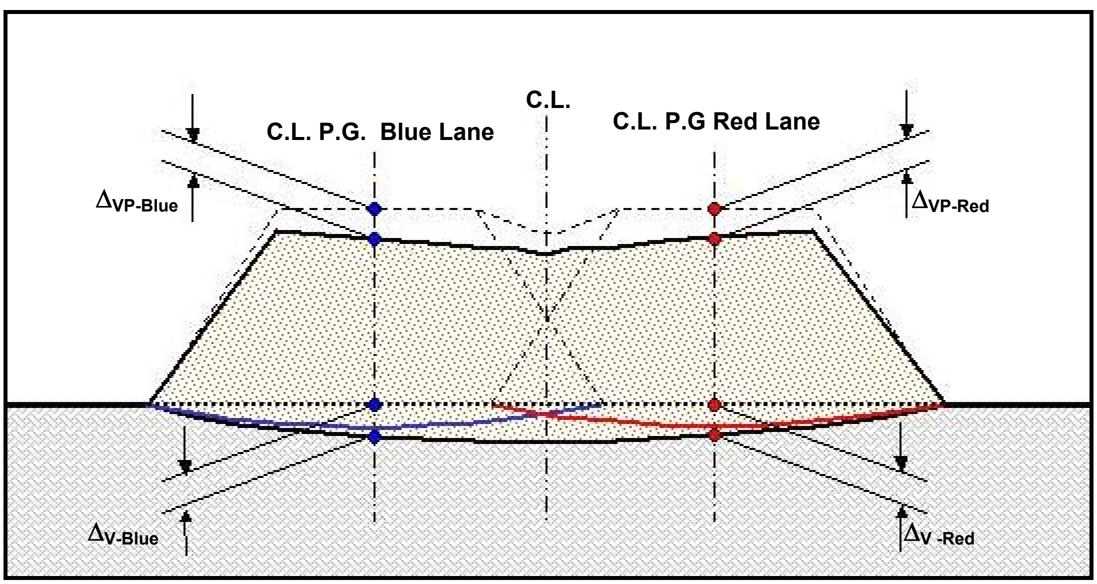


Figure 10-4, Divided Highway (Section A-A)

The Performance Limit EV-01 is for maximum settlement (Δ_V) that occurs at the profile grade over the design life of the embankment that begins after the pavement has been placed. The Performance Limit EV-02 is the maximum settlement rate that occurs after paving along the profile grade. The maximum settlement rate is specified as a constant rate of settlement that is allowed per year after the roadway has been paved.

Performance Limit EV-03 is specified as the maximum differential settlement (δ_V) occurring longitudinally along the profile grade. The differential settlement is specified over a distance of 50 feet, measured longitudinally along the embankment. If vertical displacements are encountered at an isolated location such as shown in Figure 10-5, the differential settlement performance limit EV-03 may be pro-rated so that at any point along the distance, L, the tolerances specified are not exceeded. There are no Performance Limits for differential settlements (δ_V) that occur perpendicular (transverse) to the alignment for new embankments since these displacements are relatively small due to the relatively uniform loading and the assumed low soil variability in the transverse direction not typically investigated. If transverse differential settlement is anticipated, such as is observed during a roadway widening, refer to Section 10.4.4.

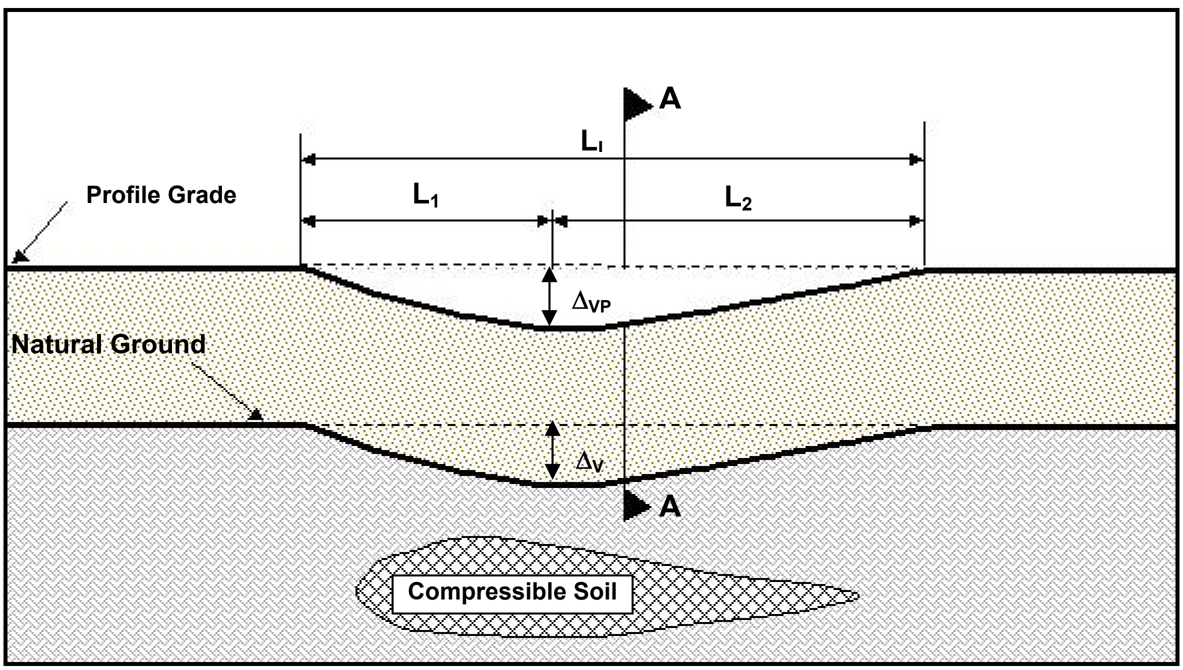


Figure 10-5, Embankment Settlement Profile

10.4.4 Transverse Differential Embankment Settlements

Existing embankments are often widened to accommodate additional traffic lanes or are widened in order to accommodate a re-alignment of a new bridge being constructed adjacent to an existing bridge. These Performance Limits are used on roadways where differential settlement due to widening of the roadway or to soil variability could adversely affect the roadway pavement. The embankment subject to transverse differential embankment settlement shall be designed for the Performance Limits indicated in Table 10-7 (EV-01, EV-02, and EV-03), and transverse differential embankment settlement Performance Limit provided in Table 10-8.

Table 10-8, Embankment Widening Settlement Performance Limits

Notation	Deformation ID No.	Description
Settlement, Δ_V	**EV-01**	Maximum Settlement from Elastic Compression + Primary Consolidation along the profile grade (Δ_{VP}) over the design life of the embankment. The design life begins after the pavement has been placed.
	EV-02	Maximum Settlement Rate from Primary Consolidation + Secondary Consolidation per year after the roadway has been paved.
Differential Settlement, δ_V	**EV-03**	Maximum Differential Settlement from Primary Consolidation + Secondary Consolidation occurring longitudinally along the profile grade after the roadway has been paved
	EV-04	Maximum Differential Settlement occurring transverse to the profile grade after the roadway has been paved

When existing embankments are widened, a parallel profile grade is established at the location of maximum vertical settlement for the embankment widening as shown in Figure 10-6. Figure 10-6 is designated as Section A-A that corresponds to an embankment widening cross-section taken transverse to the travel lane as indicated in Figure 10-5. The performance limits, EV-01, EV-02, and EV-03, are computed in the same manner as discussed in section 10.4.3 except that the settlements are computed along the profile of maximum settlement, Δ_{VT}. The maximum vertical differential settlement (EV-04) limits the differential settlements between the existing embankment and the embankment widening section that may affect the paved roadway surface. The differential settlements transverse to the embankment is computed at distance "L_T" between the existing embankment (where zero or minimal settlement occurs) and the new embankment at point of maximum settlement as indicated in Figure 10-6.

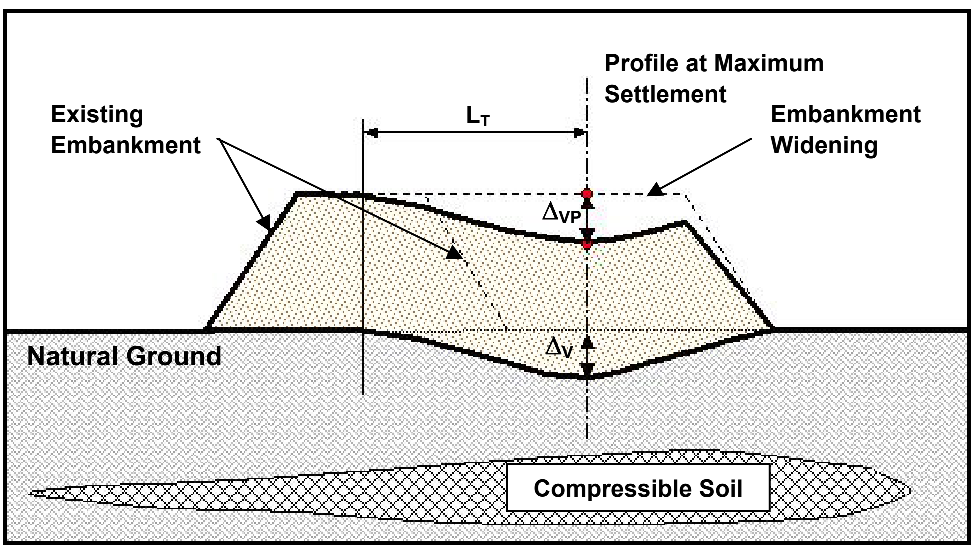


Figure 10-6, Embankment Widening Settlement (Section A-A)

10.4.5 Embankment/Bridge Transition Settlement

At the transition between the bridge approach embankments and the bridge ends there is a potential for large differential vertical settlement (δ_V). The vertical differential settlement can be significant in magnitude because the bridge end bents are typically supported on deep foundations that are relatively stationary in the vertical direction as compared to the approach embankment. If the new bridge approach embankments are placed over compressible soils the approach embankments tend to settle significantly more than the bridge ends. Performance Limits for the Embankment/Bridge transition settlement are identified in Table 10-9.

Table 10-9, Bridge/Embankment Transition Settlement Performance Limits

Notation	Deformation ID No.	Description
Vertical Differential Settlement, δ_V	**EV-05**	Maximum Differential Settlement (δ_V) between the bridge End Bent and the end of the Approach Slab after the roadway has been paved.

Differential vertical settlements between the bridge ends and the approach embankments can significantly affect the roadway rideability at the bridge abutment and at the end of the approach slab as shown in Figure 10–7.

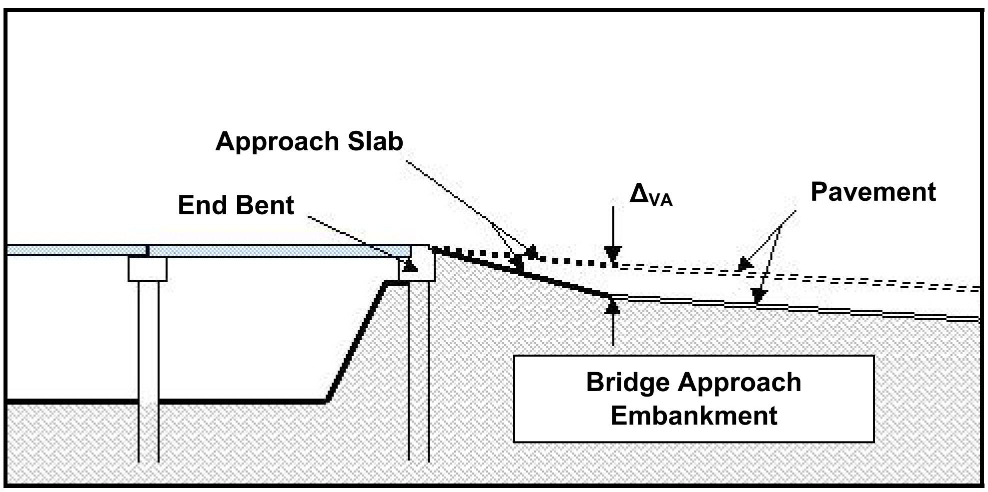


Figure 10-7, Bridge Approach Embankment Settlement

Performance Limit EV-05 is specified as a percentage of the length of the approach slab (L_{Slab}) in feet. The differential settlement (δ_V) is the absolute value of the difference between the settlement at the end of the approach slab (Δ_{VA}) and the settlement at the End Bent (Δ_{VE}). The vertical settlement at the End Bent (Δ_{VE}) is discussed in Section 10.5.2. The Performance Limit at the Service limit state is used to minimize the displacements typically observed at the bridge ends that are typically referred to as the “bump at the end of the bridge.” The Extreme Event I limit state performance limit is used to maintain Damage and Service Levels required for the design earthquake.

10.5 BRIDGE DEFORMATIONS

10.5.1 Bridge Terminology and Deformation Notations

The design of bridge deep foundations is discussed in Chapter 16. Bridge terminology used to specify geotechnical performance limits for bridge foundations is presented in Table 10-10. For more discussion of the terminology in Table 10-10, prefer to the *Bridge Design Manual*. In case of conflicts with the terminology in the *Bridge Design Manual*, the *Bridge Design Manual* takes precedence for this table only.

Table 10-10, Bridge Terminology

Terminology	Description
Bent	The bridge substructure that supports the bridge superstructure at intervals along the bridge superstructure.
End Bent	The bridge substructure that supports the bridge superstructure at the bridge abutments. This type of structure has three configurations that affect the deformations of the bridge • Integral • Semi-Integral • Free Standing
Integral End Bent	Superstructure extends into the end wall and the end wall is rigidly connected to the pile cap.
Semi-Integral End Bent	Similar to the Integral End Bent except a bond breaker is placed between the end wall and the pile cap and the beams rest on a bearing.
Free Standing End Bent	Superstructure supported by bearings on pile cap with end wall separating superstructure from fill.
Interior Bent	The bridge substructure that supports the bridge superstructure at intervals between the ends of the bridge (End Bents).
Span	The center-to-center distance between bridge supports (Bents). This term is also sometimes used to refer to the bridge superstructure located between supports. The bridge superstructure typically consists of either beams, girders, slabs, trusses, etc.
End Span	The center-to-center distance between the support at the end of the bridge (End Bent) and the first or last interior bridge support (1^{st} or last Interior Bent), at either end of the bridge.
Interior Span	The center-to-center distance between two interior bridge supports (Interior Bents).
Simple Span Bridge	A bridge comprised of one or more spans where the superstructure is not connected between adjacent spans. A load applied in one span will not produce any effects on the other spans.
Continuous Span Bridge	A bridge comprised of several spans where the superstructure is fully connected between adjacent spans and a load applied in one span produces an effect on the other spans.
Bridge Deck	The vehicle riding platform (typically reinforced concrete) that distributes the traffic live loads to the beams, girders, trusses, etc. of the bridge superstructure.

Typical bridge terminology is depicted in Figure 10-8.

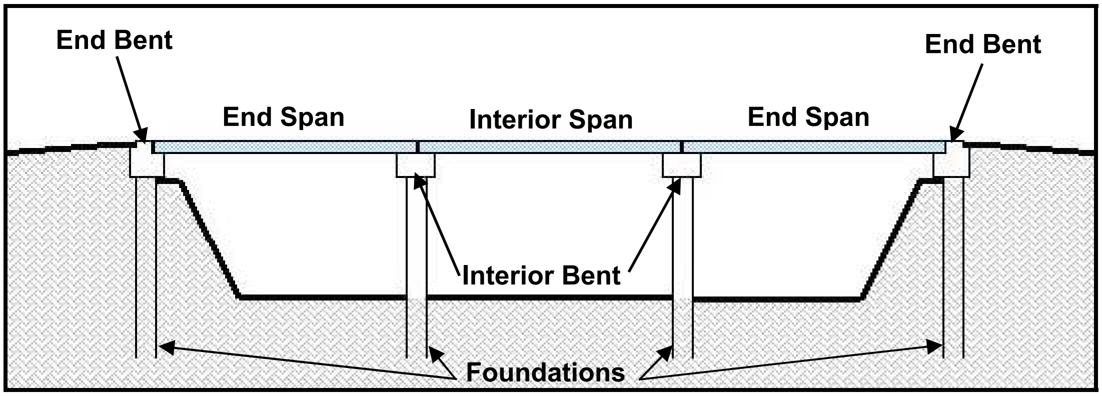


Figure 10-8, Bridge Layout (Simple or Continuous Span)

Vertical deformations are evaluated at the centerline (C.L.) of the bridge structure which typically coincides with the bridge Profile Grade (P.G.) elevation. The performance limits assume a uniform settlement across each individual bent support in the transverse direction. Design of bridge foundations should not allow transverse differential settlement within an end bent or interior bent. Adjustments in the location where vertical deformations are measured may be made in order to evaluate the maximum deformations that the bridge may undergo. Bridge deformation notations are listed in Table 10-11.

Table 10-11, Bridge Deformation Notations

Notation	Description
δ_V	Vertical Differential Settlement
Δ_{VE}	Vertical Settlement at End Bent (Abutment)
Δ_{VI}	Vertical Settlement at Interior Bent
Δ_L	Lateral Displacement
Δ_{LL}	Lateral Displacement in Longitudinal direction
Δ_{LT}	Lateral Displacement in Transverse direction
L_{Span}	Center-to-center distance between bridge supports (End Span or Interior Span)

The bridge foundation deformations can be described by the following categories:

- End Bent Vertical Deformation (Section 10.5.2)
- Interior Bent Vertical Deformation (Section 10.5.3)
- Lateral Deformations (Section 10.5.4)

The performance limits provided in the following sections are independent of the type of foundation and are dependent on the bridge deformations that occur as a result of the bridge supports. Typically either driven piles or drilled shafts are used as foundations. In some circumstances spread footings may be allowed. Deformation descriptions are the same for simple and continuous span bridges. The analyses of continuous bridges can be more complex and is discussed in Section 10.5.5.

10.5.2 End Bent Vertical Deformations

End bent deformation at bridge abutments is sometimes due to instability of the approach embankments as shown in Figure 10–9. See Section 10.4.2 for more information concerning global instability deformations. End bent deformations may also occur as a result of foundation displacement due to seismic hazards (liquefaction, lateral spreading, etc.), collisions, downdrag forces, foundation settlement, and weak foundation support. Performance limits for end bent vertical deformation are identified in Table 10-12.

Table 10-12, End Bent Vertical Deformation Performance Limits

Notation	Deformation ID No.	Description
Vertical Differential Settlement, δ_V	**EB-01**	Maximum Vertical Differential Settlement (Δ_{VE}) between an **Integral/Semi-Integral** End Bent and the first Interior Bent.
	EB-02	Maximum Vertical Differential Settlement (Δ_{VE}) between a **Free Standing** End Bent and the first Interior Bent.

The Performance Limit (EB-01 and EB–02) for maximum vertical differential settlement (δ_V) between the end bent and the first interior bent is specified as a ratio of the length of the end span ($L_{Span} = L_{End\ Span}$). The vertical differential settlement (δ_V) is the absolute value of the difference between the vertical settlement at the end bent, Δ_{VE}, (Figure 10-9) and the vertical settlement of the first interior bent, Δ_{VI} (see Figure 10–10).

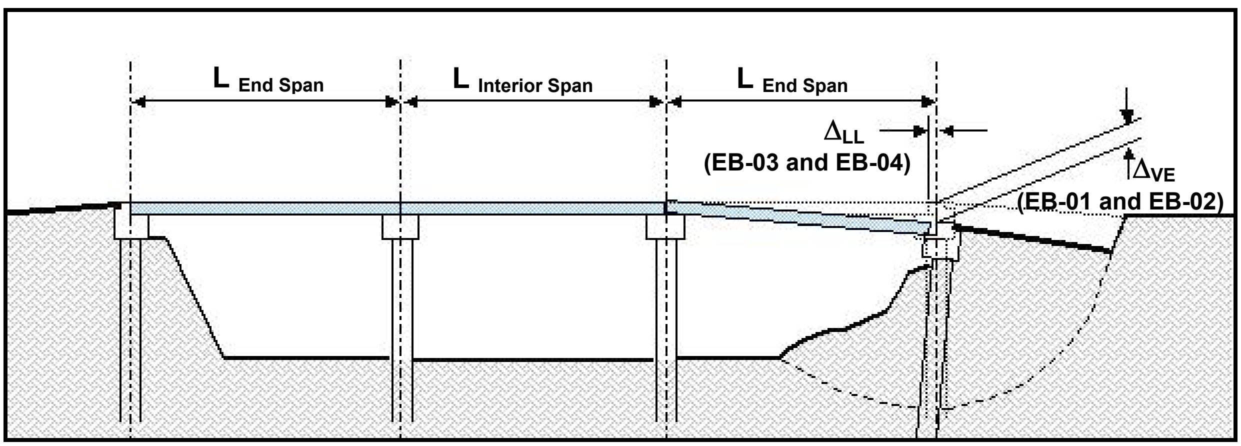


Figure 10-9, Bridge End Bent Slope Instability Deformation

10.5.3 Interior Bent Vertical Deformations

Interior bent deformations can occur as a result of foundation displacement due to seismic hazards (Liquefaction, Lateral Spreading, etc.), downdrag forces, foundation settlement, and weak foundation support. Although slope instability affecting interior bridge bents is rare, interior bridge bents can be affected by slope instability and should therefore be analyzed when appropriate. Performance limits for interior bent vertical deformation are identified in Table 10-13.

Table 10-13, Interior Bent Vertical Deformation Performance Limits

Notation	Deformation Limit ID No.	Description
Vertical Differential Settlement, δ_V	**IB-01**	Maximum Vertical Differential Settlement (Δ_{VI}) for **Integral/Semi-Integral** Interior Bent.
	IB-02	Maximum Vertical Differential Settlement (Δ_{VI}) for **Free Standing** Interior Bent.

The Performance Limit (IB-01 and IB–02) for maximum vertical differential settlement (δ_V) between interior bents and adjacent bents is specified as a ratio of the length of the adjacent spans of the interior bent being analyzed. The span length in feet is determined by using the center-to-center span length (L_{Span}) between each adjacent bent. Since interior bents have a span on each side, the performance limit and differential settlement should be computed for each adjacent span to insure that all Performance Limits are met. The vertical differential settlement (δ_V) is the absolute value of the difference between the vertical settlement of the interior bent, Δ_{VI} (see Figure 10-10), being analyzed and the vertical settlement of the adjacent bent. If the first interior bent on the right side of Figure 10-10 is being evaluated the performance limits would need to be evaluated for a span to the right of the bent ($L_{Span} = L_{End\ Span}$) and for the span to the left of the bent ($L_{Span} = L_{Interior\ Span}$).

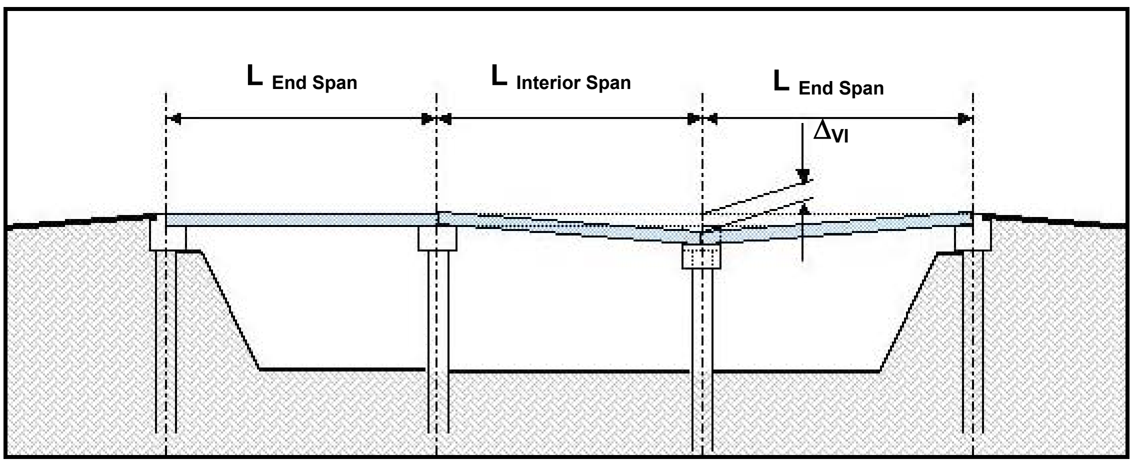


Figure 10-10, Bridge Interior Bent Settlement

10.5.4 Lateral Deformations

Lateral displacements are typically due to lateral loadings being exerted on the foundation elements or bridge abutments. Lateral loadings are typically exerted during Extreme Events resulting from seismic hazards or collisions, but may be caused by traffic on bridges with horizontal curves. Bridge approach embankment instability discussed in Section 10.4.2 can also exert lateral forces at the bridge end bents (abutment). Lateral displacements can be critical since excessive displacements can lead to collapse of a bridge by damaging bridge bearings and/or by causing structural damage to the foundations. Performance Limits for end bent and interior bent lateral deformation are identified in Table 10-14 and Table 10-15, respectively.

Table 10-14, End Bent Lateral Deformation Performance Limits

Notation	Deformation ID No.	Description
Lateral Longitudinal Displacement, Δ_{LLE}	**EB-03**	Maximum Lateral Longitudinal Displacement for **Integral/Semi-Integral** End Bent (Δ_{LLE})
	EB-04	Maximum Lateral Longitudinal Displacement for **Free Standing** End Bent (Δ_{LLE})
Lateral Transverse Displacement, Δ_{LTE}	**EB-05**	Maximum Lateral Transverse Displacement for **Integral/Semi-Integral** End Bent (Δ_{LTE})
	EB-06	Maximum Lateral Transverse Displacement for **Free Standing** End Bent (Δ_{LTE})

Table 10-15, Interior Bent Lateral Deformation Performance Limits

Notation	Deformation Limit ID No.	Description
Lateral Longitudinal Displacement, Δ_{LLI}	**IB-03**	Maximum Lateral Longitudinal Displacement for **Integral/Semi-Integral** Interior Bent (Δ_{LLI})
	IB-04	Maximum Lateral Longitudinal Displacement for **Free Standing** Interior Bent (Δ_{LLI})
Lateral Transverse Displacement, Δ_{LTI}	**IB-05**	Maximum Lateral Transverse Displacement for **Integral/Semi-Integral** Interior Bent (Δ_{LTI})
	IB-06	Maximum Lateral Transverse Displacement for **Free Standing** Interior Bent (Δ_{LTI})

Lateral displacements (Δ_L) in the longitudinal (Δ_{LL}) and transverse (Δ_{LT}) directions for interior bents and end bents are indicated in Figure 10-11. The performance limits for lateral displacement in the longitudinal and transverse direction are provided as either numerical values in inches or as a percentage of the height, H, in feet, from the top of footing or point of fixity of driven pile/drilled shaft to the top of bent cap.

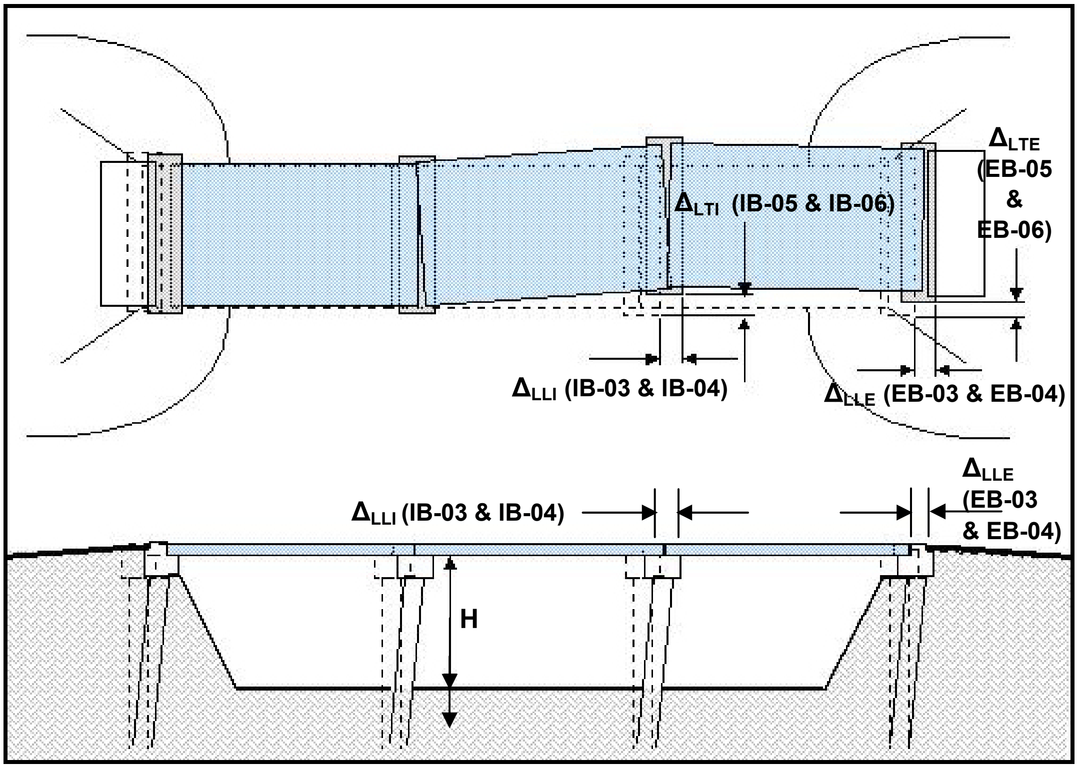


Figure 10-11, Bridge Lateral Displacement

10.5.5 Continuous Bridge Deformations

Continuous span bridges such as shown in Figure 10-12 will deform similarly to simple span bridges. The main difference is that because the structure is continuous, the structural behavior of the bridge will be more complex. Vertical deformations in this type of structure tend to induce stresses over the bridge supports (bents) that are considerably higher than if it were a simply supported bridge. This behavior makes it more critical to accurately predict deformations for continuous structures since higher stresses may lead to structural damage at the bridge supports that would then increase the stresses in the bridge superstructure. Lateral deformations also tend to induce larger stresses at the bridge supports than for simply supported structures.

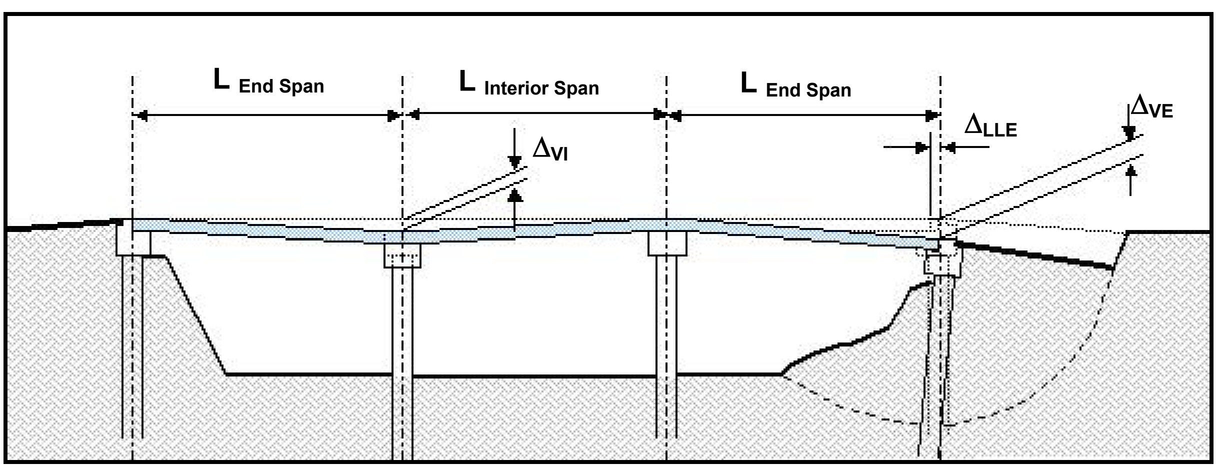


Figure 10-12, Continuous Bridge Settlements

10.6 EARTH RETAINING STRUCTURE DEFORMATIONS

10.6.1 Earth Retaining Structure Terminology and Deformation Notations

Earth retaining structure selection and design are discussed in Chapter 18. For the purposes of defining Performance Limits, Earth Retaining Structures (ERS) have been classified based on the retained soil being in-place (Cut ERS) or the retained soil being placed during construction (Fill ERS). Cut ERS refers to a retaining system that is constructed from the top of the wall to the base concurrent with excavation operations of the in-place soil being retained. Fill ERS refers to a retaining system that is constructed from the base of the wall to the top and placing the retained soil during construction. Terminology used to specify geotechnical performance limits for earth retaining structures is presented in Table 10-16.

Table 10-16, Earth Retaining Structures Terminology

Terminology	Description
Earth Retaining Structure (ERS)	An engineered structural system that prevents the lateral advance of a soil mass by resisting the lateral earth pressures exerted by the soil. Earth retaining structures have been classified for Strength limit state design by the type of retaining system as follows: • Rigid Gravity ERS • Flexible Gravity ERS • Cantilever ERS Performance limits for Earth Retaining Structures are provided based on the retained soil being in-place (Cut ERS) as indicated in Table 10-17 or the retained soil being placed during construction (Fill ERS) as indicated in Table 10-18.
Gravity ERS	An ERS that prevents the advance of select fill materials placed during construction and is constructed from the base of the wall to the top. Fill ERS can be used in Cut situations, provided that the retained soil adjacent to the wall construction can be stabilized during construction by either cutting back the retained soil on a slope or by using temporary shoring to retain the soil. Gravity retaining walls can be either rigid or flexible, depending on the wall system.
Rigid Gravity ERS	Rigid gravity walls are typically fill ERS that have rigid facings and rigid structural elements such as those used in Standard Brick Walls, Concrete Retaining Walls
Flexible Gravity ERS	Flexible gravity walls are typically fill ERS that have flexible facings and flexible structural elements such as those used in Gabion Wall, Crib Wall, Bin Wall, MSE (Modular Block Facing), MSE (Precast Panel Facing), MSE (Gabion Facing), and Geosynthetic Reinforced Soil Slopes.
Cantilever ERS	An ERS that prevents the advance of an in-situ soil mass and is typically constructed from the top of the wall to the base concurrent with excavation operations of the in-place soil to be retained. Cantilever retaining ERS can either be constructed with or without tie-back anchors. Typical cantilever ERS used are Sheet Pile Wall, Soldier Pile Wall, Tangent/Secant Pile Wall, Soldier Pile Wall w/ Anchor, Tangent/Secant Pile Wall w/ Anchors, and Soil Nailed Wall.
ERS Profile	A profile of the wall that indicates the top of the wall, the location where the wall intersects the natural ground, and the bottom of the wall (embedment depth of the wall below natural ground). Wall profiles typically have their own alignment and stationing and are tied in to the project alignment.
ERS Cross-Section	A slice or section taken perpendicular to the wall profile at a specific location (station).

Cut ERS and Fill ERS that are commonly used by SCDOT have been grouped by categories as indicated in Tables 10-17 and 10-18, respectively.

Table 10-17, Cut – Earth Retaining Structures (ERS)

Category	Type
Cantilever Walls	Sheet Pile Wall, Soldier Pile Wall, Tangent/Secant Pile Wall
Cantilever Walls with Anchors	Soldier Pile Wall w/ Anchor, Tangent/Secant Pile Wall w/ Anchors
In-Situ Reinforced Earth Walls	Soil Nailed Wall

Table 10-18, Fill – Earth Retaining Structures (ERS)

Wall Type	Category	Type
Rigid Gravity Walls	**Rigid/Semi-Rigid Gravity Walls**	Standard Brick Walls, Concrete Barrier Walls, Concrete Retaining Walls
Flexible Gravity Walls	**Prefabricated Modular Gravity Wall**	Gabion Wall, Crib Wall, Bin Wall
	Mechanically Stabilized Earth Walls	MSE (Modular Block Facing) MSE (Precast Panel Facing) MSE (Gabion Facing)
	Reinforced Soil Slope	Geosynthetic Reinforced Soil Slopes

The performance limits for Cut and Fill earth retaining structures are based on the intended use of the wall and the type of wall. There are many types of walls and each wall has its own limitations, advantages, and disadvantages with respect to economics, construction, and performance. Proper ERS selection is essential for the retaining system to meet the performance limits required. Unless otherwise indicated, the deformations that are described in this section apply to both cut and fill type earth retaining structures. Earth retaining structure deformation notations are listed in Table 10-19.

Table 10-19, ERS Deformation Notations

Notation	Description
δ_V	Vertical Differential Settlement
Δ_V	Vertical Settlement
Δ_{VTW}	Vertical Settlement at Top of Wall at a specific location along the wall profile
Δ_{VBW}	Vertical Settlement at Bottom of Wall or where embedded walls intersect the natural ground at a specific location along the wall profile
Δ_{VTS}	Vertical Displacement at the Top of the Slope failure surface
Δ_{VBS}	Vertical Displacement at the Bottom of the Slope failure surface
Δ_{VR}	Maximum Vertical Displacement of soil reinforcement
δ_L	Lateral Differential Displacement along the top of the wall
Δ_L	Lateral Displacement
Δ_{LTW}	Lateral Displacement at Top of Wall at a specific location along the wall profile
Δ_{LBW}	Lateral Displacement at the Bottom of the Wall or where embedded walls intersect the natural ground at a specific location along the wall profile
Δ_{LTS}	Lateral Displacement at the Top of the Slope failure surface
Δ_{LBS}	Lateral Displacement at the Bottom of the Slope failure surface
θ	Angle of rotation after slope instability or settlement deformations have occurred
ΔL	Deformation occurring along the critical failure surface due to slope instability
L	Distance used to denote boundaries for differential settlement computations

The performance limits for earth retaining structures are specified for the following types of deformations:

- Global Instability Deformations (Section 10.6.2)
- Longitudinal Settlement Deformation (Section 10.6.3)
- Transverse Settlement Deformation (Section 10.6.4)
- Lateral Displacements (Section 10.6.5)

Methods to evaluate stability and deformations are provided in Chapters 13 and 17.

10.6.2 Global Instability Deformations

Earth retaining structures are subject to global instability deformations similar to roadway embankments. For an in-depth discussion of global instability deformations see Section 10.4.2. Performance Limits for earth retaining structures due to slope instability deformations are identified in Table 10-20.

Table 10-20, ERS Performance Limits for Slope Instability

Notation	Deformation ID No.	Description
Lateral Displacement, Δ_L	**RS-01**	Maximum Lateral Displacement (Δ_{LTW}) at the top of the wall.
Vertical Displacement, Δ_V	**RS-02**	Maximum Vertical Displacement (Δ_{VTW}) at the top of the wall.
Wall Rotation, θ	**RS-03**	Wall rotation is a measure of center verticality. The angle of rotation of the ERS Facing after slope instability deformations have occurred. A positive (+) angle indicates that the wall has rotated inward, towards the retained soil. A negative (-) angle indicates that the wall has rotated outward away from the retained soil.

The Performance Limit (RS-01) is the maximum lateral displacement that occurs at the top of the wall as a result of the global instability deformations as shown in Figure 10-13. The Performance Limit (RS-02) is the maximum differential vertical displacement along the top of the wall (longitudinally) as indicated in Figures 10-14 and 10-15. The Performance Limit (RS-03) is the effective wall tilt or rotation and is measured as the angle between the original wall face and the rotated wall face.

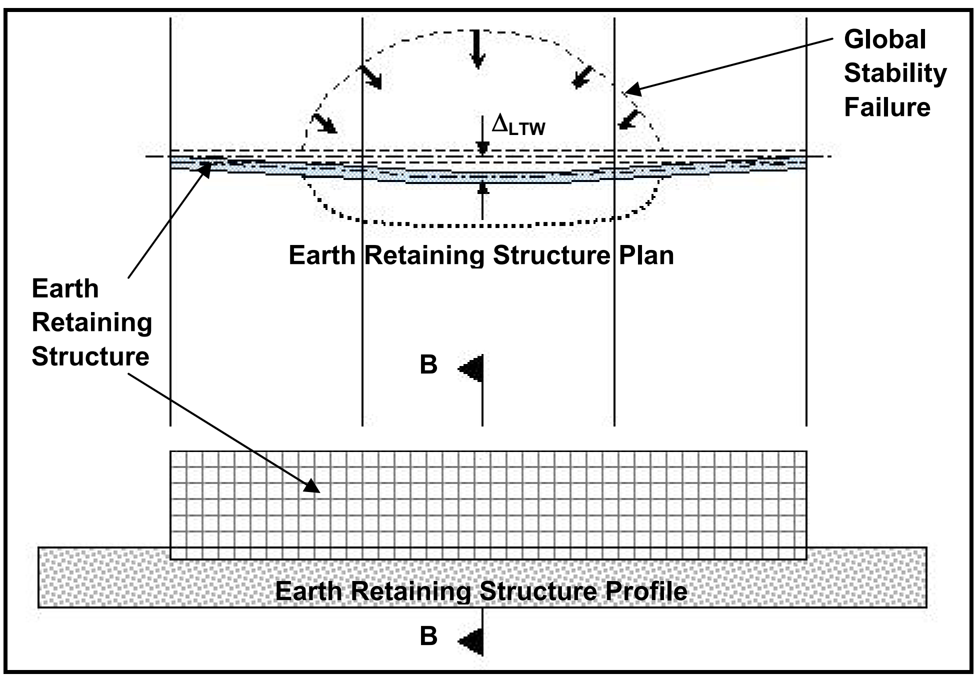


Figure 10-13, ERS Global Instability

Section B-B indicated in Figure 10-13 is shown for global instability resulting from circular-arc and sliding-wedge failure surfaces in Figures 10-14 and 10-15, respectively.

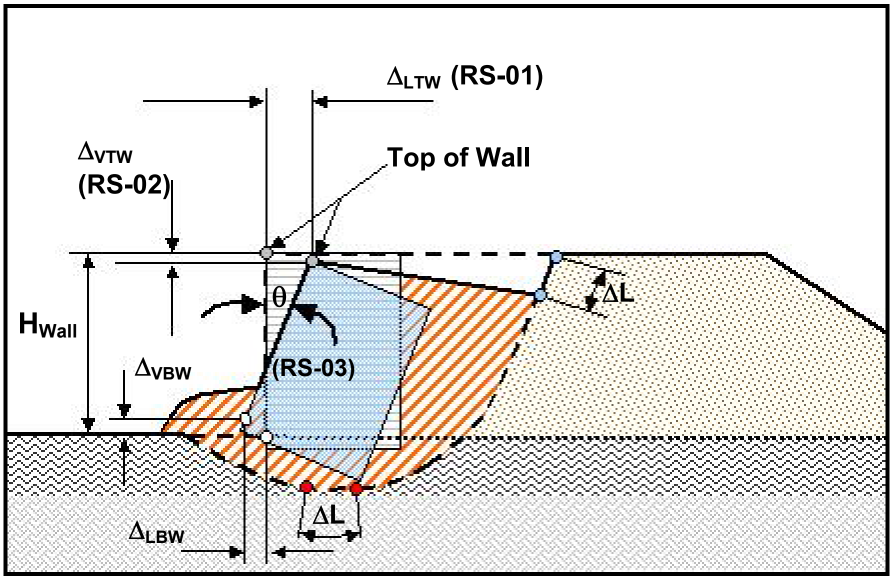


Figure 10-14, ERS Circular-Arc Instability (Section B-B)

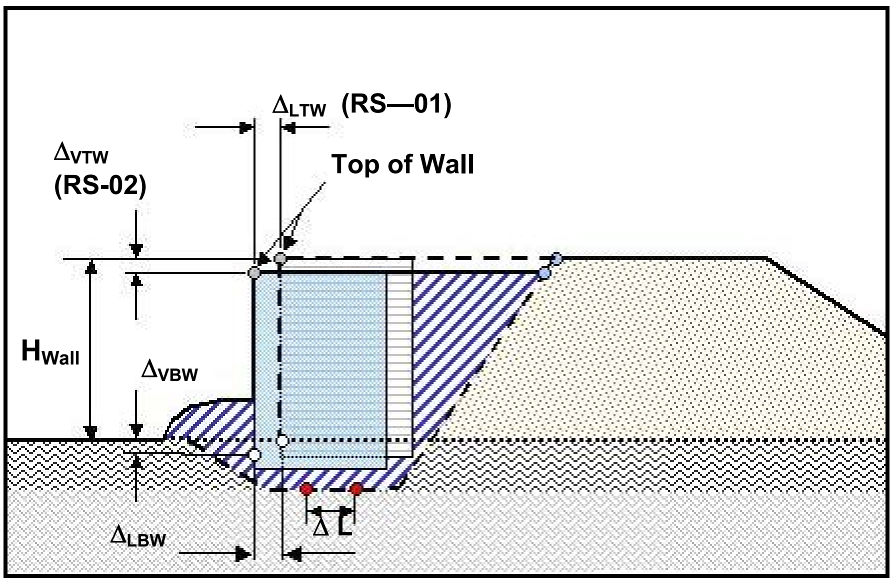


Figure 10-15, ERS Sliding-Wedge Instability (Section B-B)

10.6.3 Settlement Deformation - Longitudinal

ERS settlements are typically due to fill walls placed over compressible soils. This type of settlement is typically due to elastic compression and consolidation (primary and secondary) of the compressible soils. ERS settlements can also be due to seismic hazards such as liquefaction of the subgrade during or after a seismic event. ERS settlements are evaluated at the top of the wall adjacent to the wall facing where differential settlements are likely to cause the most distress to the wall facing. Performance Limits for settlements occurring longitudinally (along the wall profile) are identified in Table 10-21. Methods to evaluate settlements are provided in Chapters 13 and 17.

Table 10-21, ERS Settlement (Longitudinal) Performance Limits

Notation	Deformation Limit ID No.	Description
Vertical Settlement, Δ_V	**RV-01**	Maximum Vertical Settlement at the top of wall profile grade (Δ_{VTW}) over the design life of the embankment.
	RV-02	Maximum Settlement Rate per year after the wall has been constructed.
Vertical Differential Settlement, δ_V	**RV-03**	Maximum Vertical Differential Settlement observed longitudinally along the top of wall profile grade after the wall has been constructed.

The Performance Limit (RV-01) is the maximum settlement that occurs at the face at the top of the wall profile over the design life of the ERS as indicated in Figure10-16. The Performance Limit (RV-02) is a maximum rate of settlement that occurs after wall facing is constructed along the top of the wall profile. The rate of settlement is measured as the settlement occurring per year after the wall facing has been constructed.

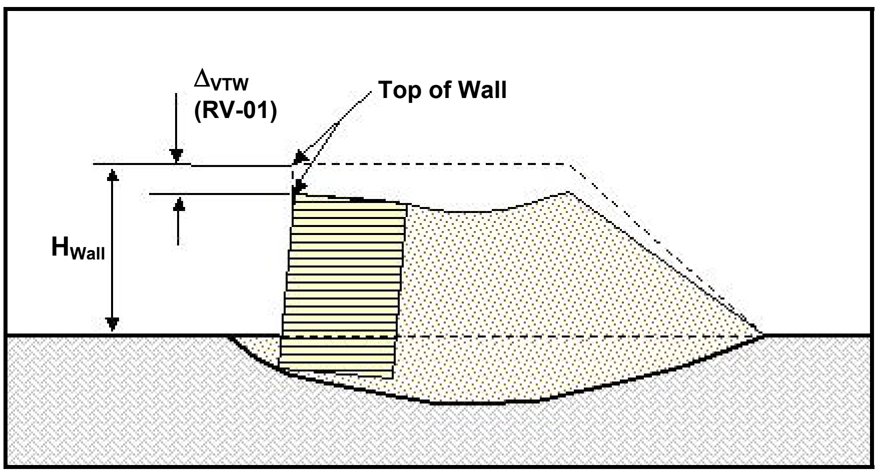


Figure 10-16, ERS Settlement (Section B–B)

Wall distress due to settlements along the top of wall profile, Δ_{VTW}, are limited by specifying a Performance Limit (RV-03) for the maximum differential settlement (δ_V) observed longitudinally along the top of wall profile after the ERS has been constructed. The differential settlement is specified over a distance of 50 feet, measured longitudinally along the top of wall profile. If vertical displacements are encountered at an isolated location such as shown in Figure 10–17, the differential settlement Performance Limit (RV-03) may be pro-rated so that at any point along the distance, L_S, the tolerances specified are not exceeded.

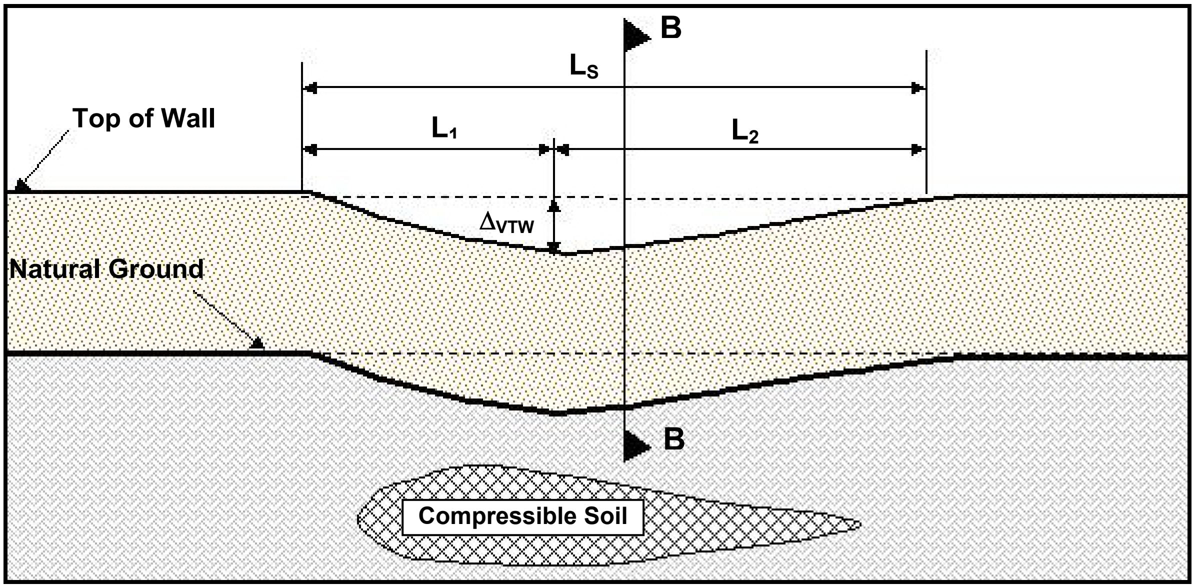


Figure 10-17, ERS Settlement Profile

10.6.4 Settlement Deformation - Transverse

This Performance Limit is used for differential settlements (δ_V) that occur perpendicular to the wall alignment and is only applicable to retaining walls that have discrete soil reinforcements (geosynthetic reinforcement, steel reinforcement, soil anchors, etc.) extending perpendicular to the wall facing to the end of the length of the reinforcement, L. The Performance Limit for settlement occurring perpendicular to the wall profile (transverse direction) is identified in Table 10-22.

Table 10-22, ERS Settlement (Transverse) Performance Limits

Notation	Deformation Limit ID No.	Description
Vertical Differential Settlement, δ_V	**RV-04**	Maximum Vertical Differential Settlement observed perpendicular (transverse) to the top of wall profile after the wall has been constructed.

Examples of ERS with reinforced soil (MSE walls) and ERS with tieback anchors (cantilever walls w/ tieback anchors) are shown in Figures 10-18 and 10-19, respectively. Excessive differential settlements (transverse) may cause distress and even wall collapse from the added load induced to the wall facing and soil reinforcements. The Performance Limit (RV-04) is the maximum differential settlements perpendicular (transverse) to the adjusted profile over a distance, L_R, as indicated in Figure 10-18 and 10-19. Performance Limit (RV-04) is computed along maximum increments of 5 feet.

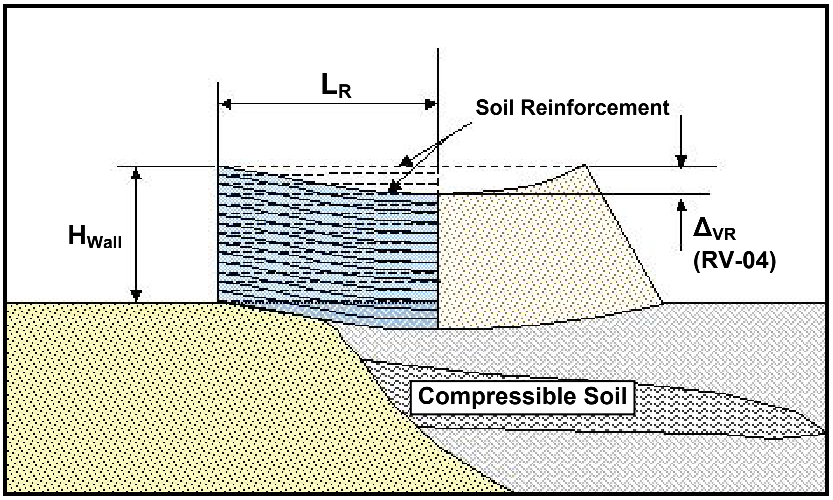


Figure 10-18, ERS Reinforced Soils - Transverse Differential Settlement

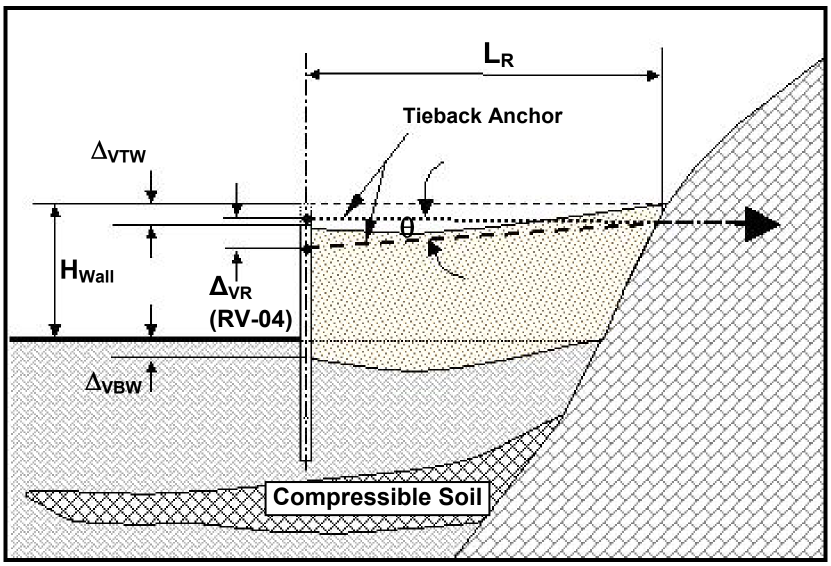


Figure 10-19, ERS Tieback Anchor - Transverse Differential Settlement

10.6.5 Lateral Displacements

ERS lateral displacements are those movements that occur as a result of lateral soil pressures. Lateral soil pressure loadings produce displacements of the structural members of the wall system and also displacements of the soil (soil-structure interaction). ERS lateral displacements can also occur as a result of active seismic loadings that are transmitted laterally to the earth retaining structure. The Performance Limits for lateral displacements occurring perpendicular to the wall profile (transverse direction) are identified in Table 10-23.

Table 10-23, ERS Lateral Performance Limits

Notation	Deformation ID No.	Description
Lateral Displacement, Δ_L	**RL-01**	Maximum Lateral Displacement (Δ_{LTW}) at the top of the wall.
Wall Rotation, θ	**RL-02**	Angle of rotation of the ERS Facing after slope instability deformations have occurred. A positive (+) angle indicates that the wall has rotated inward, towards the retained soil. A negative (-) angle indicates that the wall has rotated outward away from the retained soil.
Lateral Differential Displacement, δ_L	**RL-03**	Maximum Differential Lateral Displacement (Δ_{LTW}) longitudinally along the top of the wall. This performance limit is typically referred to as wall "bulging."

The Performance Limit (RL-01) is the maximum lateral displacement that occurs at the top of the wall over the design life of the structure. The Performance Limit (RL-02) is the effective wall tilt or rotation and is measured as the angle between the original wall face and the rotated wall face. ERS Performance Limit (RL-01) and (RL-02) are evaluated at the top of the wall and also as the effective wall rotation or tilt as indicated in Figures 10-20 and 10-21.

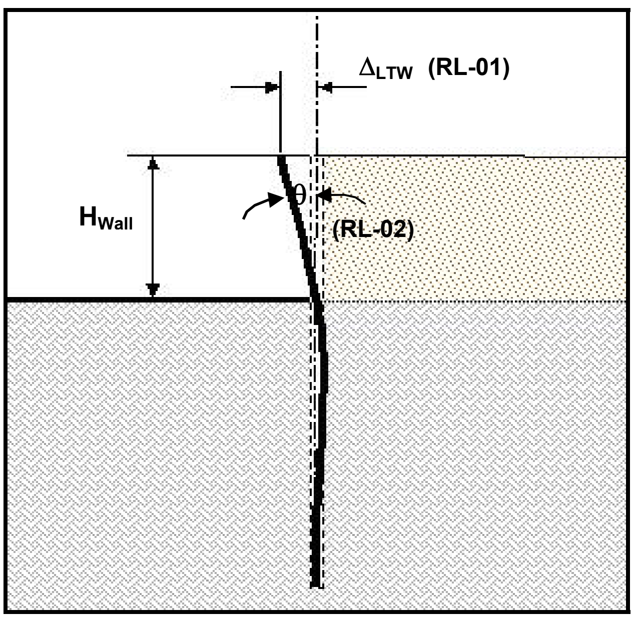


Figure 10-20, Cut ERS Section C-C Lateral Deformations

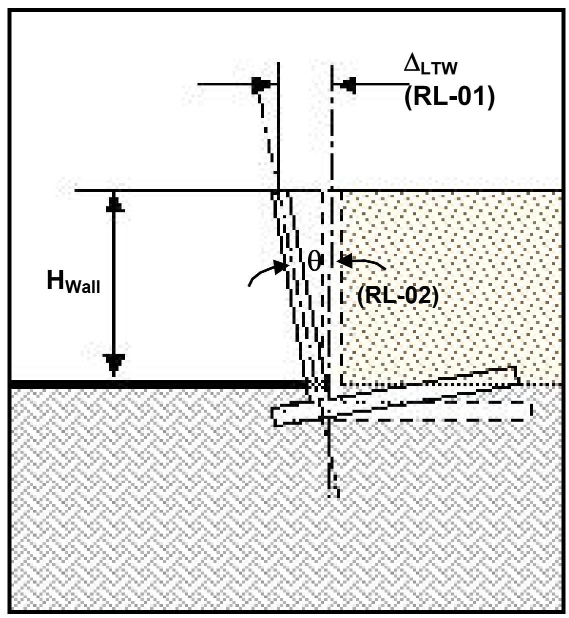


Figure 10-21, Fill ERS Section C-C Lateral Deformations

Lateral wall distress (bulging), due to differential lateral displacement along the top of wall profile, Δ_{LTW}, are limited by specifying a Performance Limit (RL-03) for the maximum differential lateral displacement (Δ_L) observed longitudinally along the top of wall profile after the ERS has been constructed as shown in Figure 10-22. The differential lateral displacement is specified over a distance of 50 feet and measured longitudinally along the top of wall profile. If vertical displacements are encountered at an isolated location, the differential settlement Performance Limit (RL-03) may be pro-rated so that at any point along the distance, L_L, the tolerances specified are not exceeded.

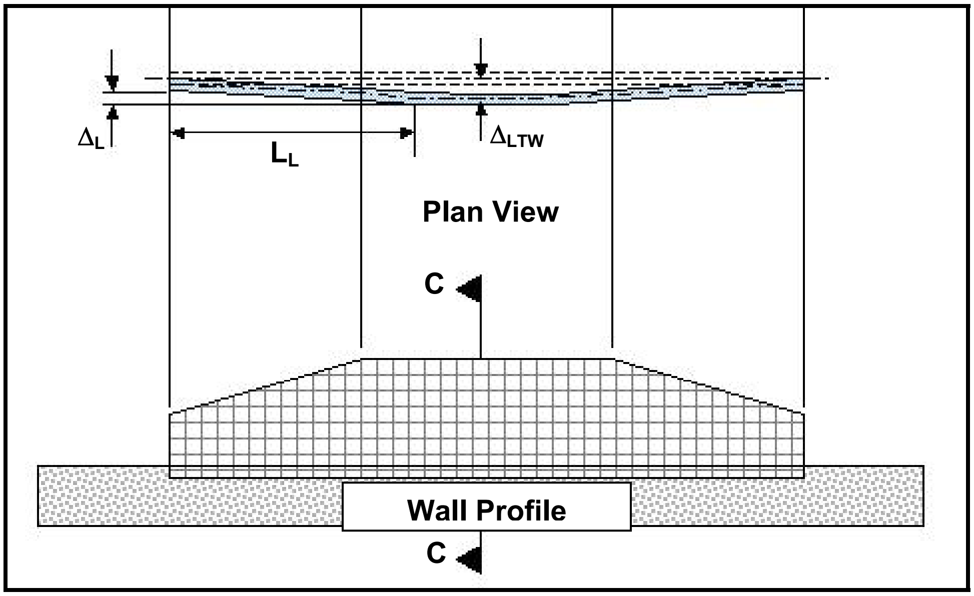


Figure 10-22, ERS Lateral Deformations

10.7 PERFORMANCE LIMITS FOR EMBANKMENTS

10.7.1 Service Limit State

10.7.1.1 Performance Objective

The Performance Objectives for embankments at the Service limit state (SLS) are that the embankment remains fully functional for the design life of the structure and that through periodic maintenance any deformations can be adjusted to maintain the serviceability requirements of the roadway pavement. See Section 10.2.1 for additional requirements that were used to develop the Performance Limits.

10.7.1.2 Performance Limits

The following embankment performance limits have been developed to meet the Performance Objective indicated in Section 10.7.1.1. These embankment performance limits have been classified based on the Roadway Structure Operational Classification (ROC) described in Chapter 8. Embankment deformation descriptions are found in Section 10.4.

Table 10-24, Embankment Performance Limits at SLS

Deformation ID No.		Service Limit State Performance Limit Description	ROC I	ROC II	ROC III
		Minimum Design Life (Years)	100	100	100
Settlement (Longitudinal)	EV-01	Maximum Vertical Settlement along the profile grade over the design life of the embankment. (Inches)	8.00”	8.00”	16.00”
	EV-02	Maximum Settlement Rate per year after the roadway has been paved. (Inches per year)	0.10	0.10	0.20
	EV-03	Maximum Vertical Differential Settlement occurring longitudinally along the profile grade after the roadway has been paved. Differential ratio is shown in parenthesis for informational purposes. (Inches per 50 Feet of Embankment Longitudinally)	1.00” (1/600)	1.50” (1/400)	2.00” (1/300)

Table 10-25, Bridge/Embankment Transition Settlement Performance Limit at SLS

Deformation ID No.		Service Limit State Performance Limit Description	ROC I	ROC II	ROC III
Settlement (Longitudinal)	EV-05	Maximum Vertical Differential Settlement Between End Bent and End of Approach Slab (Inches). The Approach Slab length (L_{Slab}) is measured in feet.	$0.075 \times L_{Slab}$	$0.100 \times L_{Slab}$	$0.125 \times L_{Slab}$

Table 10-26, Embankment Widening Performance Limits at SLS

Deformation ID No.		Service Limit State Performance Limit Description	ROC I	ROC II	ROC III
		Minimum Design Life (Years)	100	100	100
Settlement (Longitudinal)	EV-01	Maximum Vertical Settlement at the adjusted profile grade over the design life of the embankment. (Inches)	8.00”	8.00”	16.00”
	EV-02	Maximum Settlement Rate per year after the roadway has been paved. (Inches per year)	0.10	0.10	0.20
	EV-03	Maximum Vertical Differential Settlement occurring longitudinally along the adjusted profile grade after the roadway has been paved. Differential ratio is shown in parenthesis for informational purposes. (Inches per 50 Feet of Embankment Longitudinally)	1.00” (1/600)	1.50” (1/400)	2.00” (1/300)
Settlement (Transverse)	EV-04	Maximum Vertical Differential Settlement occurring transverse to the adjusted profile grade between the existing embankment and the new widened embankment after the roadway has been paved. (Inches per 5 feet of embankment width)	0.10” (1/600)	0.15” (1/400)	0.20” (1/300)

10.7.2 Extreme Event I Limit State

10.7.2.1 Performance Objective

Performance Objectives for embankments after an Extreme Event I (EE I) has occurred are provided in Table 10-27. These Performance Objectives are based solely on the embankment providing support for the roadway pavement and maintaining the road open to traffic. Descriptions of Service and Damage performance levels are provided in Section 10.2.2.

Table 10-27, Embankment Extreme Event I Performance Objectives

<table>
<tr><th rowspan="2">Design Earthquake</th><th rowspan="2">Performance Level</th><th colspan="3">ROC</th></tr>
<tr><th>I</th><th>II</th><th>III</th></tr>
<tr><td rowspan="2">Functional Evaluation Earthquake (FEE)</td><td>Service</td><td>Immediate</td><td>Maintained</td><td>Recoverable</td></tr>
<tr><td>Damage</td><td>Minimal</td><td>Repairable</td><td>Repairable</td></tr>
<tr><td rowspan="2">Safety Evaluation Earthquake (SEE)</td><td>Service</td><td>Maintained</td><td>Impaired</td><td>Impaired</td></tr>
<tr><td>Damage</td><td>Repairable</td><td>Significant</td><td>Significant</td></tr>
</table>

10.7.2.2 Performance Limits

Table 10-28, Embankment Global Instability Performance Limits at EE I Limit State

<table>
<tr><th colspan="2" rowspan="2">Deformation ID No.</th><th rowspan="2">EE I Limit State Performance Limit Description [1]</th><th rowspan="2">Design EQ</th><th colspan="3">ROC</th></tr>
<tr><th>I</th><th>II</th><th>III</th></tr>
<tr><td rowspan="4">Vertical Displacement</td><td rowspan="2">GI-01</td><td rowspan="2">Maximum Vertical Displacement at top of the slope failure surface. (Inches)</td><td>FEE</td><td>1.00”</td><td>2.00”</td><td>4.00”</td></tr>
<tr><td>SEE</td><td>2.00”</td><td>4.00”</td><td>8.00”</td></tr>
<tr><td rowspan="2">GI -02</td><td rowspan="2">Maximum Vertical Displacement at bottom of the slope failure surface. (Inches)</td><td>FEE</td><td>1.00”</td><td>2.00”</td><td>4.00”</td></tr>
<tr><td>SEE</td><td>2.00”</td><td>4.00”</td><td>8.00”</td></tr>
<tr><td rowspan="4">Lateral [2] Displacement</td><td rowspan="2">GI-03</td><td rowspan="2">Maximum Lateral Displacement at top of the slope failure surface. (Inches)</td><td>FEE</td><td>3.00”</td><td>6.00”</td><td>24.00”</td></tr>
<tr><td>SEE</td><td>4.00”</td><td>12.00”</td><td>60.00”</td></tr>
<tr><td rowspan="2">GI-04</td><td rowspan="2">Maximum Lateral Displacement at bottom of the slope failure surface. (Inches)</td><td>FEE</td><td>3.00”</td><td>6.00”</td><td>24.00”</td></tr>
<tr><td>SEE</td><td>4.00”</td><td>12.00”</td><td>60.00”</td></tr>
</table>

(1) Project specific requirements may need to be selected for these performance limits if adjacent structures require more restrictive deformations. The geotechnical and structural engineers should evaluate these performance limits to determine applicability to the specific project.

(2) In the direction of global instability.

Table 10-29, Embankment Settlement Performance Limits at EE I Limit State

Deformation ID No.		EE I Limit State Performance Limit Description	Design EQ	ROC I	ROC II	ROC III
Settlement (Longitudinal)	EV-03	Maximum Vertical Differential Settlement occurring longitudinally along the profile grade after the roadway has been paved. Differential ratio is shown in parenthesis for informational purposes. (Inches per 50 Feet of Embankment Longitudinally)	FEE	1.00" (1/600)	1.50" (1/400)	2.00" (1/300)
			SEE	2.00" (1/300)	3.00" (1/200)	4.00" (1/150)

Table 10-30, Bridge/Embankment Transition Settlement Performance Limit EE I LS

Deformation ID No.		EE I Limit State Performance Limit Description	Design EQ	ROC I	ROC II	ROC III
Settlement (Longitudinal)	EV-05	Maximum Vertical Differential Settlement Between End Bent and End of Approach Slab (Inches) The Approach Slab length (L_{Slab}) is measured in feet.	FEE	0.075 L_{Slab}	0.100 L_{Slab}	0.125 L_{Slab}
			SEE	0.100 L_{Slab}	0.200 L_{Slab}	0.400 L_{Slab}

Table 10-31, Embankment Widening Settl. Performance Limits at EE I Limit State

Deformation ID No.		EE I Limit State Performance Limit Description	Design EQ	ROC I	ROC II	ROC III
Settlement (Longitudinal)	EV-03	Maximum Vertical Differential Settlement occurring longitudinally along the profile grade after the roadway has been paved. Differential ratio is shown in parenthesis for informational purposes. (Inches per 50 Feet of Embankment Longitudinally)	FEE	1.00" (1/600)	1.50" (1/400)	2.00" (1/300)
			SEE	2.00" (1/300)	4.00" (1/150)	8.00" (1/75)
Settlement (Transverse)	EV-04	Maximum Vertical Differential Settlement occurring perpendicular to the adjusted profile grade between the existing embankment and the new widened embankment after the roadway has been paved. (Inches per 5 feet of embankment width)	FEE	0.10" (1/600)	0.15" (1/400)	0.20" (1/300)
			SEE	0.20" (1/300)	0.40" (1/150)	1.00" (1/60)

10.8 PERFORMANCE LIMITS FOR BRIDGES

10.8.1 Service Limit State

10.8.1.1 Service Limit State Performance Objective

The Performance Objectives for bridges at the Service limit state (SLS) are that they remain fully functional to normal traffic for the life of the structure. Additional requirements that were used to develop the Performance Limits are provided in Section 10.2.1. Performance limits for bridge foundations are based on the bridge superstructure requirements.

10.8.1.2 Service Limit State Performance Limits

The following Performance Limits have been developed to meet the Performance Objectives indicated in Section 10.8.1.1. Deformation descriptions are found in Section 10.5.

Table 10-32, Bridge Performance Limits at SLS

Deformation ID No.		Service Performance Limit Performance Limit Description	OC		
			I	II	III
		Design Life (Years)	75	75	75
Bridge End Bents	EB-01	Maximum Vertical Differential Settlement for **Integral/Semi-Integral** End Bent (Inches) (1)	0.020 L_{Span}	0.020 L_{Span}	0.020 L_{Span}
	EB-02	Maximum Vertical Differential Settlement for **Free Standing** End Bent (Inches) (1)	0.040 L_{Span}	0.040 L_{Span}	0.040 L_{Span}
	EB-03	Maximum Lateral Longitudinal Displacement for **Integral/Semi-Integral** End Bent (Inches)	0.25”	0.50”	0.50”
	EB-04	Maximum Lateral Longitudinal Displacement for **Free Standing** End Bent (Inches)	0.50”	0.75”	0.75”
	EB-05	Maximum Lateral Transverse Displacement for **Integral/Semi-Integral** End Bent (Inches)	0.50”	0.50”	0.50”
	EB-06	Maximum Lateral Transverse Displacement for **Free Standing** End Bent (Inches)	0.75”	0.75”	0.75”
Bridge Interior Bents	IB-01	Maximum Vertical Differential Settlement for **Fixed Bearing** Interior Bent (Inches) (1)	0.020 L_{Span}	0.020 L_{Span}	0.020 L_{Span}
	IB-02	Maximum Vertical Differential Settlement for **Expansion Bearing** Interior Bent (Inches) (1)	0.040 L_{Span}	0.040 L_{Span}	0.040 L_{Span}
	IB–03	Maximum Lateral Longitudinal Displacement for **Fixed Bearing** Interior Bent (Inches)	0.50”	0.75”	0.75”
	IB–04	Maximum Lateral Longitudinal Displacement for **Expansion Bearing** Interior Bent (Inches)	0.75”	1.00”	1.00”
	IB–05	Maximum Lateral Transverse Displacement for **Fixed Bearing** Interior Bent (Inches)	0.75”	0.75”	0.75”
	IB–06	Maximum Lateral Transverse Displacement for **Expansion Bearing** Interior Bent (Inches)	1.00”	1.00”	1.00”

(1) Where L_{Span} is the center-to-center span length measured in feet. Where L_{Span} is the center-to-center distance of the first interior span adjacent to the end bent. For interior bents, L_{Span} is the shortest center-to-center span length between adjacent bridge spans.

10.8.2 Extreme Event I Limit State

10.8.2.1 Extreme Event I Limit State Performance Objective

Even though bridges may suffer damage and may need to be replaced after a seismic event, all bridges (regardless of their Bridge Classification) will be designed for no-collapse due to earthquake shaking and geologic seismic hazards (i.e. liquefaction) associated with the design earthquake. In order for a bridge to satisfy the no-collapse requirement, bridges must remain supported throughout the seismic event.

Extreme Event I Performance Objectives are expressed in terms of Service Levels and Damage Levels. Performance Objectives for bridge foundations are based on the bridge superstructure requirements. Service Levels and Damage Levels descriptions are provided in Section 10.2.2. These levels provide an assessment of how the bridge will perform after an earthquake. Even though these Performance Objectives are subjective, they are the basis for developing Performance Limits for bridges subjected to Extreme Event I loading conditions. This limit state requires that bridge foundations be designed for the FEE and SEE Design Event Earthquakes. Performance Objectives for the overall Service and Damage Levels of the Bridge System have been developed as indicated in Table 10–33.

Table 10-33, Bridge System Extreme Event I (Seismic) Performance Objectives (Modified SCDOT Seismic Specifications for Highway Bridges, 2008)

Design Earthquake	Performance Level	Bridge Operational Classification (OC)		
		I	II	III
Functional Evaluation Earthquake (FEE)	Service	Immediate	Maintained	Impaired [1]
	Damage	Minimal	Repairable	Significant [1]
Safety Evaluation Earthquake (SEE)	Service	Maintained	Impaired	Impaired
	Damage	Repairable	Significant	Significant

[1] The *SCDOT Seismic Specifications for Highway Bridges* (2008) do not include FEE design earthquake performance objectives for bridges with OC= III because structural analyses are only required for the SEE design earthquake. The implied FEE Performance Objective for bridges with an importance classification of OC=III is therefore the same as for the SEE design earthquake. Geotechnical analyses for roadway structures (embankment, ERS) are required for both the FEE and SEE design earthquakes regardless of bridge importance classification.

The bridge system consists of various units including superstructure, connection components, restraint components, capacity protected components, and substructure (foundations). Performance Objectives for Damage Levels of the various Bridge Components have also been established as shown in Table 10–34.

Only Performance Limits for the bridge substructure (foundations) will be addressed in this Manual. The Performance Objectives for the Superstructure are provided in order to give the designer a better understanding of the overall performance that the bridge designer is seeking.

Table 10-34, Bridge Components Damage Level Objectives
(Modified *SCDOT Seismic Specifications for Highway Bridges*, 2008)

Bridge Component		Design Earthquake	Bridge Operational Classification (OC)		
			I	II	III
Superstructure		FEE	Minimal	Minimal	Minimal [4]
		SEE	Minimal	Minimal	Minimal
Connection Components [1]		FEE	Repairable	Repairable	Significant [4]
		SEE	Significant	Significant	Significant
Interior Bent Restraint Components [2]		FEE	Minimal	Minimal	Minimal [4]
		SEE	Minimal [5]	Minimal [5]	Minimal [5]
End Bent Restraint Components [2]		FEE	Minimal	Minimal	Significant [4]
		SEE	Significant	Significant	Significant
Capacity Protected Components [3]		FEE	Minimal	Minimal	Minimal [4]
		SEE	Minimal	Minimal	Minimal
Substructure	Single Column Bents	FEE	Minimal	Repairable	Significant [4]
		SEE	Repairable	Significant	Significant
	Multi Column Bents	FEE	Minimal	Repairable	Significant [4]
		SEE	Repairable	Significant	Significant
	End Bent Piles	FEE	Minimal	Repairable	Significant [4]
		SEE	Minimal	Significant	Significant
	End Bent Wing Walls	FEE	Minimal	Repairable	Significant [4]
		SEE	Significant	Significant	Significant
	Pile Bents	FEE	Minimal	Repairable	Significant [4]
		SEE	Repairable	Significant	Significant
	Pier Walls Weak Axis	FEE	Minimal	Repairable	Significant [4]
		SEE	Repairable	Significant	Significant
	Pier Walls Strong Axis	FEE	Minimal	Minimal	Repairable [4]
		SEE	Minimal	Minimal	Repairable

[1] Include Expansion Joints and Bearings
[2] Include Shear Keys, Anchor Bolts, and Dowel Bars
[3] Include Bent Cap, Footings, and Oversized Shafts
[4] The *SCDOT Seismic Specifications for Highway Bridges* (2008) do not include FEE design earthquake performance objectives for bridges with OC= III because structural analyses are only required for the SEE design earthquake. The implied FEE Performance Objective for bridges with an of OC=III is therefore the same as for the SEE design earthquake. Geotechnical analyses for roadway structures (embankment, ERS) are required for both the FEE and SEE design earthquakes regardless of bridge importance classification.
[5] Shear keys are designed not to fuse

10.8.2.2 Extreme Event I Limit State Performance Limits

Geotechnical bridge performance limits for the Extreme Event I limit state are provided for end bents and interior bents in Tables 10-35 and 10-36, respectively. The bridge performance limits included in the SCDOT *Seismic Specifications for Highway Bridges* have been used to develop the geotechnical bridge performance limits for the Extreme Event I limit state. The *Seismic Specifications for Highway Bridges* do not include FEE design earthquake performance limits for bridges with OC= III because structural analyses are only required for the SEE design earthquake. The implied FEE performance limits for bridges with an OC=III is therefore the same as for the SEE design earthquake as indicated in Tables 10-35 and 10-36. Geotechnical engineering analyses for roadway structures (embankment and ERS) are required for both the FEE and SEE design earthquakes regardless of bridge operational classification.

Table 10-35, Bridge Substructure Performance Limits at EE I Limit State (Modified *SCDOT Seismic Specifications for Highway Bridges*, 2008)

Deformation ID No.		Extreme Event I Performance Limit Description	Design EQ	OC		
				I	II	III
Bridge End Bents	EB–01	Maximum Vertical Differential Settlement for **Integral/Semi-Integral** End Bent (Inches) [1]	FEE	0.020 L_{Span}	0.020 L_{Span}	0.020 L_{Span}
			SEE	0.040 L_{Span}	0.040 L_{Span}	0.040 L_{Span}
	EB–02	Maximum Vertical Differential Settlement for **Free Standing** End Bent (Inches) [1]	FEE	0.040 L_{Span}	0.040 L_{Span}	0.040 L_{Span}
			SEE	0.080 L_{Span}	0.080 L_{Span}	0.080 L_{Span}
	EB–03	Maximum Lateral Longitudinal Displacement for **Integral/Semi-Integral** End Bent (Inches) [2]	FEE	2"	4"	12"
			SEE	4"	8"	12"
	EB–04	Maximum Lateral Longitudinal Displacement for **Free Standing** End Bent (Inches) [2]	FEE	1"	2"	8"
			SEE	3"	6"	8"
	EB–05	Maximum Lateral Transverse Displacement **Integral/Semi-Integral** End Bent (Inches) [2]	FEE	2"	4"	12"
			SEE	4"	8"	12”
	EB–06	Maximum Lateral Transverse Displacement for **Free Standing** End Bent (Inches) [2]	FEE	2"	4"	12"
			SEE	4"	8"	12”

(1) Where L_{Span} is the center-to-center distance of the end span adjacent to the end bent measured in feet.

(2) Performance limits for lateral displacements are provided at the top of the bent cap.

Table 10-36, Bridge Substructure Performance Limits at EE I Limit State (SCDOT Seismic Specifications for Highway Bridges, 2008)

Deformation ID No.		Extreme Event I Performance Limit Description	Design EQ	OC I	OC II	OC III
Bridge Interior Bents	IB-01	Maximum Vertical Differential Settlement for **Fixed Bearings** Interior Bent (Inches) [(1)]	FEE	0.020 L_{Span}	0.020 L_{Span}	0.020 L_{Span}
			SEE	0.040 L_{Span}	0.040 L_{Span}	0.040 L_{Span}
	IB–02	Maximum Vertical Differential Settlement for **Expansion Bearings** End Bent (Inches) [(1)]	FEE	0.040 L_{Span}	0.040 L_{Span}	0.040 L_{Span}
			SEE	0.080 L_{Span}	0.080 L_{Span}	0.080 L_{Span}
	IB–03	Maximum Lateral Longitudinal Displacement for Interior Bent with **Fixed Bearings** (Inches) [(2) (3)]	FEE	0.075 H	0.100 H	0.500 H
			SEE	0.300 H	0.400 H	0.500 H
	IB–04	Maximum Lateral Longitudinal Displacement for Interior Bent with **Expansion Bearings** (Inches) [(2) (3)]	FEE	0.050 H	0.075 H	0.400 H
			SEE	0.200 H	0.300 H	0.400 H
	IB–05	Maximum Lateral Transverse Displacement for Interior Bent (Inches) [(2) (3)]	FEE	0.075 H	0.100 H	0.500 H
			SEE	0.250 H	0.400 H	0.500 H

(1) Where L_{Span} is the center-to-center span length measured in feet. For interior bents, L_{Span} is the shortest center-to-center span length between adjacent bridge spans.
(2) Performance limits for lateral displacements are provided at the top of the bent cap. The variable "H" is the height in feet from the top of bent cap to the top of footing or point of fixity of drilled shaft/driven pile.
(3) The maximum lateral longitudinal displacements may be increased provided that it does not exceed 75 percent of the bearing area at interior bents.

10.9 PERFORMANCE LIMITS FOR EARTH RETAINING STRUCTURES

10.9.1 Service Limit State

10.9.1.1 Service Limit State Performance Objective

The Performance Objectives for Earth Retaining Structures (ERS) at the Service limit state (SLS) are that they remain fully functional for the design life of the structure and that through periodic maintenance any deformations can be adjusted to maintain the serviceability and design requirements of the earth retaining structure. See Section 10.2.1 for additional requirements that were used to develop the Performance Limits.

10.9.1.2 Service Limit State Performance Limits

Geotechnical Performance Limits have been developed for Fill Earth Retaining Structures (ERS) and Cut Earth Retaining Structures (ERS) in Tables 10-37 and 10-38, respectively. These Performance Limits have been developed to meet the Performance Objective indicated in Section 10.9.1.1. ERS deformation descriptions are defined in Section 10.6.

Table 10-37, Fill ERS Performance Limits at SLS

Deformation ID No.		Service Limit State Performance Limit Description		ROC I	ROC II	ROC III
		Minimum Design Life (Years) [1]		100	100	75
Settlement (Longitudinal)	RV-01	Maximum Vertical Settlement at any point on top of the wall profile grade over the design life of the ERS (Inches)		12.00”	12.00”	18.00”
	RV-02	Maximum Rate of Settlement per year after the ERS has been constructed (Inches per year)		0.10	0.10	0.20
	RV-03	Maximum Vertical Differential Settlement at Top of Wall Profile grade over the life of the structure. (Inches/50 feet along the length of ERS) (Maximum settlement ratio indicated in parenthesis for informational purposes only)	Rigid/Semi-Rigid walls [2]	1.00” (1/600)	1.25” (1/500)	1.25” (1/500)
			Full Height Panel Facing	1.00” (1/600)	1.25” (1/500)	1.25” (1/500)
			Crib Wall, Bin Wall	1.50” (1/400)	2.00” (1/300)	2.50” (1/240)
			MSE Panel Facing Joint Spacing < ½”	1.50” (1/400)	2.00” (1/300)	3.00” (1/200)
			MSE Panel Facing Joint Spacing ≥ ½”	2.00” (1/300)	3.00” (1/200)	4.00” (1/150)
			MSE Block Facing	2.50” (1/240)	2.50” (1/240)	3.00” (1/200)
			Gabion Facing, Reinforced Soil Slope	6.00” (1/100)	12.00” (1/50)	15.00” (1/40)
Settlement (Transverse)	RV-04	Maximum Vertical Differential Settlement Perpendicular to the wall facing profile over the design life of the structure. [3] (Inches/5 feet perpendicular to wall or slope face)	MSE Walls	0.150 L_{Reinf}	0.150 L_{Reinf}	0.150 L_{Reinf}
			Reinforced Soil Slopes	0.150 L_{Reinf}	0.150 L_{Reinf}	0.150 L_{Reinf}
Lateral Displacements	RL-01	Maximum Lateral Displacement at the top of the wall. [4] (Inches)	Rigid Walls, Full Height Panel Facing	0.015 H_{Wall}	0.015 H_{Wall}	0.025 H_{Wall}
			Crib Wall, Bin Wall, MSE Walls	0.035 H_{Wall}	0.035 H_{Wall}	0.045 H_{Wall}
			Gabion Facing, Reinforced Soil Slope	0.050 H_{Wall}	0.050 H_{Wall}	0.060 H_{Wall}
	RL-02	Maximum Differential Lateral Displacement longitudinally along the top of the wall. (Inches/50 feet of wall)	All Earth Retaining Structures	1.00”	1.00”	1.00”
	RL-03	Maximum Tilt or Angle of Rotation (θ) of the ERS Facing from the original constructed ERS facing after lateral displacements have occurred. (Degrees)	Rigid Walls, Full Height Panel Facing	0 - 0.5°	0 - 0.5°	0 - 0.5°
			Crib Wall, Bin Wall, MSE Walls	<2°	<2°	<2°
			Gabion Facing, Reinforced Soil Slope	<3°	<3°	<3°

(1) The Minimum Design Life for temporary structures that will be in service more than 3 years is 75 years. The Minimum Design Life for temporary earth retaining structures that will be in service for less than 3 years is 3 years.

(2) Rigid/Semi-Rigid retaining walls include reinforced concrete walls and brick walls.

(3) The soil reinforcement length (L_{Reinf}) is measured in feet.

(4) The wall height (H_{Wall}) is measured in feet. For the reinforced soil slopes the H_{Wall} is the vertical distance from the toe of the slope to shoulder edge.

Table 10-38, Cut ERS Performance Limits at SLS

Deformation ID No.		Service Limit State Performance Limit Description		ROC I	ROC II	ROC III
		Minimum Design Life (Years) [1]		100	100	75
Settlement (Longitudinal)	**RV-01**	Maximum Vertical Settlement at any point on top of the wall profile grade over the design life of the ERS (Inches)		8.00”	8.00”	8.00”
	RV-02	Maximum Rate of Settlement per year after the ERS has been constructed (Inches per year)		0.10	0.10	0.20
	RV-03	Maximum Vertical Differential Settlement at Top of Wall Profile grade over the life of the structure. (Inches/50 feet of wall) (Maximum settlement ratio indicated in parenthesis for informational purposes only)	All Cut Earth Retaining Structures	1.50” (1/400)	2.00” (1/300)	3.00” (1/200)
Settlement (Transverse)	**RV-04**	Maximum Vertical Differential Settlement Perpendicular to the wall facing profile over the design life of the structure. [2] (Inches/5 feet of wall)	Embedded Walls w/Anchors, In-Situ Reinforced Earth Walls	0.100 L_{Anchor}	0.100 L_{Anchor}	0.150 L_{Anchor}
Lateral Displacements	**RL-01**	Maximum Lateral Displacement at the top of the wall. [3] (Inches)	Embedded Walls w/Anchors, In-Situ Reinforced Earth Walls	0.015 H_{Wall}	0.015 H_{Wall}	0.025 H_{Wall}
			Embedded Walls	0.035 H_{Wall}	0.035 H_{Wall}	0.045 H_{Wall}
	RL-02	Maximum Differential Lateral Displacement longitudinally along the top of the wall. (Inches/50 feet of wall)	All Cut Earth Retaining Structures	1.00”	1.00”	1.00”
	RL-03	Maximum Angle of Rotation (θ) of the ERS Facing from the original constructed ERS facing after lateral displacements have occurred. (Degrees)	Embedded Walls w/Anchors, In-Situ Reinforced Earth Walls	<1°	<1°	<1°
			Embedded Walls	<2°	<2°	<2°

(1) The Minimum Design Life for temporary structures that will be in service more than 3 years is 75 years. The Minimum Design Life for temporary earth retaining structures that will be in service for less than 3 years is 3 years.
(2) The soil anchor length (L_{Anchor}) is measured in feet.
(3) The wall height (H_{Wall}) is measured in feet.

10.9.2 Extreme Event I Limit State

10.9.2.1 Performance Objective

The Performance Objectives for Earth Retaining Structures (ERS) at the Extreme Event I limit state are provided in Table 10-39. Description of Service and Damage performance levels are provided in Section 10.2.2.

Table 10-39, Embankment Extreme Event I Performance Objectives

Design Earthquake	Performance Level	Roadway Operational Classification (ROC)		
		I	II	III
Functional Evaluation Earthquake (FEE)	Service	Immediate	Maintained	Recoverable
	Damage	Minimal	Repairable	Repairable
Safety Evaluation Earthquake (SEE)	Service	Maintained	Impaired	Impaired
	Damage	Repairable	Significant	Significant

10.9.2.2 Performance Limits

Geotechnical Performance limits for Earth Retaining Structures (ERS) deformations resulting from global instability are provided in Table 10-40. Geotechnical Performance limits for Settlement of Fill Earth Retaining Structures (ERS) are provided in Table 10-41. The Geotechnical Performance Limits for ERS will typically supercede the geotechnical Performance Limits for embankments provided in Section 10.7.2. Geotechnical Performance Limits for the Extreme Event I limit state have also been developed for Fill Earth Retaining Structures (ERS) and Cut Earth Retaining Structures (ERS) in Tables 10-42 and 10-43, respectively.

Table 10-40, ERS Global Stability Performance Limits at EE I Limit State

Deformation	ID No.	Extreme Event I Limit State (EE I) Performance Limit Description			Design EQ	ROC I	ROC II	ROC III
Vertical Displacement, Δ_V	**RS-01**	Maximum Vertical Displacement at top of the slope failure surface. (Inches)			FEE	1.00”	2.00”	4.00”
					SEE	2.00”	4.00”	8.00”
Lateral Displacement, Δ_L	**RS-03**	Maximum Lateral Displacement at top of the slope failure surface. (Inches)			FEE	3.00”	6.00”	12.00”
					SEE	4.00”	12.00”	24.00”
Wall Tilt, θ	**RS-04**	Maximum angle of ERS facing tilt or rotation after slope stability deformations. (Degrees)	Fill ERS	Rigid Walls, Full Height Panel Facing	FEE	<0.5°	<0.5°	<1°
					SEE	<1°	<1°	<2°
				Crib Wall, Bin Wall, MSE Walls	FEE	<2°	<2°	<4°
					SEE	<4°	<4°	<6°
				Gabion Facing, Reinforced Soil Slope	FEE	<4°	<4°	<6°
					SEE	<6°	<6°	<8°
			Cut ERS	Embedded Walls w/Anchors, In-Situ Reinforced Earth Walls	FEE	<1°	<1°	<1°
					SEE	<2°	<2°	<2°
				Embedded Walls	FEE	<2°	<2°	<2°
					SEE	<3°	<3°	<3°

Table 10-41, Fill ERS Settlement Performance Limits at EE I Limit State

Deformation ID No.		Extreme Event I Limit State (EE I) Performance Limit Description			ROC		
					I	II	III
Settlement (Longitudinal)	RV-03	Maximum Vertical Differential Settlement at Top of Wall Profile grade over the life of the structure. (Inches/50 feet along the length of ERS) (Maximum settlement ratio indicated in parenthesis for informational purposes only)	Rigid/Semi-Rigid walls [1]	FEE	1.00” (1/600)	1.25” (1/500)	1.25” (1/500)
				SEE	2.00” (1/300)	2.50” (1/240)	2.50” (1/240)
			Full Height Panel Facing	FEE	1.00” (1/600)	1.25” (1/500)	1.25” (1/500)
				SEE	2.00” (1/300)	2.50” (1/240)	2.50” (1/240)
			Crib Wall, Bin Wall	FEE	1.50” (1/400)	2.00” (1/300)	2.50” (1/240)
				SEE	3.00” (1/200)	4.00” (1/150)	5.00” (1/480)
			MSE Panel Facing Joint Spacing < ½”	FEE	1.50” (1/400)	2.00” (1/300)	3.00” (1/200)
				SEE	3.00” (1/200)	4.00” (1/150)	6.00” (1/100)
			MSE Panel Facing Joint Spacing ≥ ½”	FEE	2.00” (1/300)	3.00” (1/200)	4.00” (1/150)
				SEE	4.00” (1/150)	6.00” (1/100)	6.00” (1/100)
			MSE Block Facing	FEE	2.50” (1/240)	2.50” (1/240)	3.00” (1/200)
				SEE	5.00” (1/480)	5.00” (1/480)	6.00” (1/100)
			Gabion Facing, Reinforced Soil Slope	FEE	6.00” (1/100)	12.00” (1/50)	12.00” (1/50)
				SEE	12.00” (1/50)	12.00” (1/50)	12.00” (1/50)
Settlement (Transverse)	RV-04	Maximum Vertical Differential Settlement Perpendicular to the wall facing profile over the design life of the structure. [2] (Inches/5 feet perpendicular to wall or slope face)	MSE Walls	FEE	0.150 L_{Reinf}	0.150 L_{Reinf}	0.150 L_{Reinf}
				SEE	0.200 L_{Reinf}	0.200 L_{Reinf}	0.200 L_{Reinf}
			Reinforced Soil Slopes	FEE	0.150 L_{Reinf}	0.150 L_{Reinf}	0.150 L_{Reinf}
				SEE	0.300 L_{Reinf}	0.300 L_{Reinf}	0.300 L_{Reinf}

(1) Rigid/Semi-Rigid retaining walls include reinforced concrete walls and brick walls.
(2) The soil reinforcement length (L_{Reinf}) is measured in feet.

Table 10-42, Fill ERS Lateral Displacement Performance Limits at EE I Limit State

Deformation ID No.		Extreme Event I Limit State (EE I) Performance Limit Description			ROC		
					I	II	III
Lateral Displacements	RL-01	Maximum Lateral Displacement at the top of the wall. [(1)] [(2)] (Inches)	Rigid Walls, Full Height Panel Facing	FEE	$0.015\ H_{Wall}$	$0.015\ H_{Wall}$	$0.025\ H_{Wall}$
				SEE	$0.030\ H_{Wall}$	$0.030\ H_{Wall}$	$0.050\ H_{Wall}$
			Crib Wall, Bin Wall, MSE Walls	FEE	$0.035\ H_{Wall}$	$0.035\ H_{Wall}$	$0.045\ H_{Wall}$
				SEE	$0.070\ H_{Wall}$	$0.070\ H_{Wall}$	$0.090\ H_{Wall}$
			Gabion Facing, Reinforced Soil Slope	FEE	$0.050\ H_{Wall}$	$0.050\ H_{Wall}$	$0.060\ H_{Wall}$
				SEE	$0.100\ H_{Wall}$	$0.100\ H_{Wall}$	$0.150\ H_{Wall}$
	RL-02	Maximum Differential Lateral Displacement longitudinally along the top of the wall. (Inches/50 feet of wall)	All Earth Retaining Structures	FEE	1.00”	1.00”	1.00”
				SEE	2.00”	2.00”	2.00”
	RL-03	Maximum Angle of Rotation (θ) of the ERS Facing from the original constructed ERS facing after lateral displacements have occurred. (Degrees)	Rigid Walls, Full Height Panel Facing	FEE	<0.5°	<0.5°	<0.5°
				SEE	<1°	<1°	<2°
			Crib Wall, Bin Wall, MSE Walls	FEE	<2°	<2°	<2°
				SEE	<4°	<4°	<6°
			Gabion Facing, Reinforced Soil Slope	FEE	<4°	<4°	<6°
				SEE	<6°	<6°	<8°

(1) Rigid/Semi-Rigid retaining walls include reinforced concrete walls and brick walls.
(2) The wall height (H_{Wall}) is measured in feet.

Table 10-43, Cut ERS Performance Limits at EE I Limit State

Deformation ID No.		Extreme Event I Limit State (EE I) Performance Limit Description			ROC I	ROC II	ROC III
Settlement (Longitudinal)	**RV-03**	Maximum Vertical Differential Settlement at Top of Wall Profile grade over the life of the structure. (Inches/50 feet of wall) (Maximum settlement ratio indicated in parenthesis for informational purposes only)	All Cut Earth Retaining Structures	FEE	1.50” (1/400)	2.00” (1/300)	3.00” (1/200)
				SEE	3.00” (1/200)	4.00” (1/150)	6.00” (1/100)
Settlement (Transverse)	**RV-04**	Maximum Vertical Differential Settlement Perpendicular to the wall facing profile over the design life of the structure. [1] (Inches/5 feet of wall)	Embedded Walls w/Anchors, In-Situ Reinforced Earth Walls	FEE	0.100 L_{Reinf}	0.100 L_{Reinf}	0.150 L_{Reinf}
				SEE	0.200 L_{Reinf}	0.200 L_{Reinf}	0.300 L_{Reinf}
Lateral Displacements	**RL-01**	Maximum Lateral Displacement at the top of the wall. [2] (Inches)	Embedded Walls w/Anchors, In-Situ Reinforced Earth Walls	FEE	0.015 H_{Wall}	0.015 H_{Wall}	0.025 H_{Wall}
				SEE	0.030H_{Wall}	0.030 H_{Wall}	0.050 H_{Wall}
			Embedded Walls	FEE	0.035 H_{Wall}	0.035 H_{Wall}	0.045 H_{Wall}
				SEE	0.070 H_{Wall}	0.070 H_{Wall}	0.090 H_{Wall}
	RL-02	Maximum Differential Lateral Displacement longitudinally along the top of the wall. (Inches/50 feet of wall)	All Cut Earth Retaining Structures	FEE	1.00”	1.00”	1.00”
				SEE	2.00”	2.00”	2.00”
	RL-03	Maximum Angle of Rotation (θ) of the ERS Facing from the original constructed ERS facing after lateral displacements have occurred. (Degrees)	Embedded Walls w/Anchors, In-Situ Reinforced Earth Walls	FEE	<1°	<1°	<1°
				SEE	<2°	<2°	<2°
			Embedded Walls	FEE	<2°	<2°	<2°
				SEE	<3°	<3°	<3°

(1) The soil reinforcement length (L_{Reinf}) is measured in feet.
(2) The wall height (H_{Wall}) is measured in feet.

10.10 REFERENCES

The geotechnical information contained in this Manual must be used in conjunction with the SCDOT *Seismic Design Specifications for Highway Bridges*, SCDOT *Bridge Design Manual*, and AASHTO LRFD Bridge Design Specifications. The Geotechnical Design Manual will take precedence over all references with respect to geotechnical engineering design.

AASHTO LRFD Bridge Design Specifications, U.S. Customary Units, 4th Edition, (2007), American Association of State Highway and Transportation Officials.

SCDOT *Bridge Design Manual* (2006), South Carolina Department of Transportation, http://www.scdot.org/doing/bridge/06design_manual.shtml

SCDOT *Seismic Design Specifications for Highway Bridges* (2008), South Carolina Department of Transportation, http://www.scdot.org/doing/bridge/bridgeseismic.shtml

Chapter 11

SOUTH CAROLINA GEOLOGY AND SEISMICITY

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Table of Contents

List of Tables

List of Figures

CHAPTER 11

SOUTH CAROLINA GEOLOGY AND SEISMICITY

11.1 INTRODUCTION

This Chapter describes South Carolina's basic geology and seismicity within the context of performing geotechnical engineering for the SCDOT. It is anticipated that the material contained in this Chapter will establish a technical framework by which basic geology and seismicity can be addressed. It is not intended to be an in-depth discussion of all the geologic formations and features found in South Carolina (SC) or a highly technical discussion of the state's seismicity. The designers are expected to have sufficient expertise in these technical areas and to have the foresight and resourcefulness to keep up with the latest advancements in these areas.

The State of South Carolina is located in the Southeastern United States and is bounded on the north by the State of North Carolina, on the west and the south by the State of Georgia, and on the east by the Atlantic Ocean. The State is located between Latitudes 32° 4' 30" N and 35° 12' 00" N and between Longitudes 78° 0' 30" W and 83° 20' 00" W. The State is roughly triangular in shape and measures approximately 260 miles East-West and approximately 200 miles North-South at the states widest points. The South Carolina coastline is approximately 187 miles long. South Carolina is ranked 40th in size with an approximate area of 30,111 square miles.

The geology of South Carolina is similar to that of the neighboring states of Georgia, North Carolina, and Virginia. These states have in the interior the Appalachian Mountains with an average elevation of 3,000 feet followed by the Appalachian Piedmont that typically ranges in elevation from 300 feet to 1000 feet. Continuing eastward from these highlands is a "Fall Line" which serves to transition into the Atlantic Coastal Plain. The Atlantic Coastal Plain gently slopes towards the Atlantic Ocean with few elevations higher than 300 feet.

The 1886 earthquake that occurred in the Coastal Plain near Charleston, South Carolina dominates the seismic history of the southeastern United States. It is the largest historic earthquake in the southeastern United States with an estimated moment magnitude, M_W, of 7.3. The damage area with a Modified Mercalli Intensity Scale of X, is an elliptical shape roughly 20 by 30 miles trending northeast between Charleston and Jedburg and including Summerville and roughly centered at Middleton Place. The intraplate epicenter of this earthquake and it's magnitude is not unique in the Central and Eastern United States (CEUS). Other intraplate earthquakes include those at Cape Ann, Massachusetts (1755) with a M_W of 5.9, and the New Madrid, Missouri (1811-1812) with M_W of at least 7.7.

The following sections describe the basic geology of South Carolina and the seismicity that will be used to perform geotechnical engineering designs and analyses. The topics discussed in these sections will be referenced throughout this Manual.

11.2 SOUTH CAROLINA GEOLOGY

South Carolina geology can be divided into three basic physiographic units: Blue Ridge Unit (Appalachian Mountains), Piedmont Unit, and the Coastal Plain Unit. The generalized locations of these physiographic units are shown in Figure 11-1.

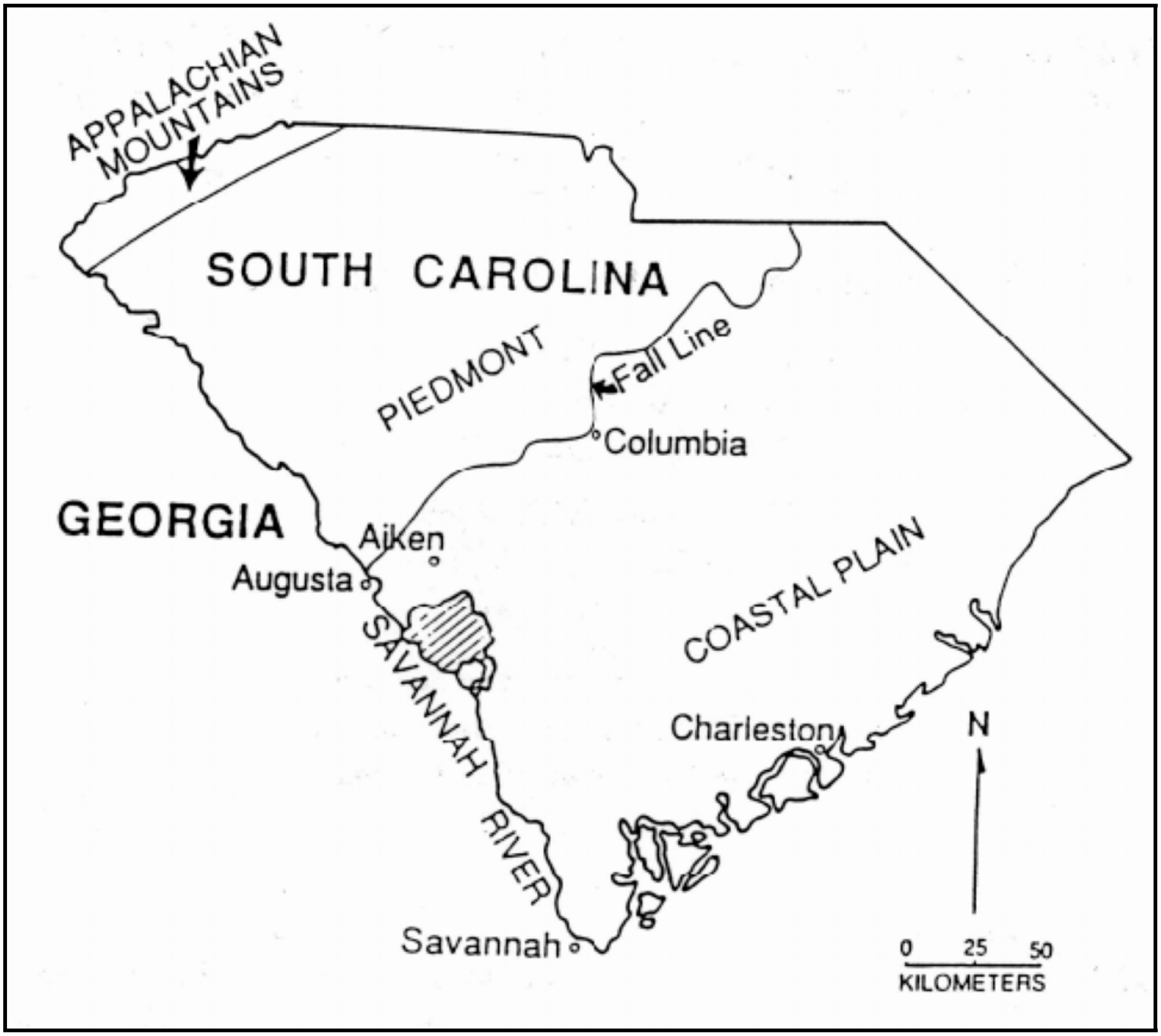


Figure 11-1, South Carolina Physiographic Units (Snipes et al., 1993)

The Blue Ridge Unit (Appalachian Mountains) covers approximately 2 percent of the state and it is located in the northwestern corner of the state. The Piedmont Unit comprises approximately one-third of the state with the Coastal Plain Unit covering the remaining two-thirds of the state. The geologic formations are typically aligned from the South-Southwest to the North-Northeast and parallel the South Carolina Atlantic coastline as shown in the generalized geologic map in Figure 11-2. The physiographic units in Figure 11-2 are broken down by the geologic time of the surface formations. South Carolina formations span in age from late Precambrian through the Quaternary period. The descriptions of events that have occurred over geologic time in South Carolina are shown in Figure 11-3.

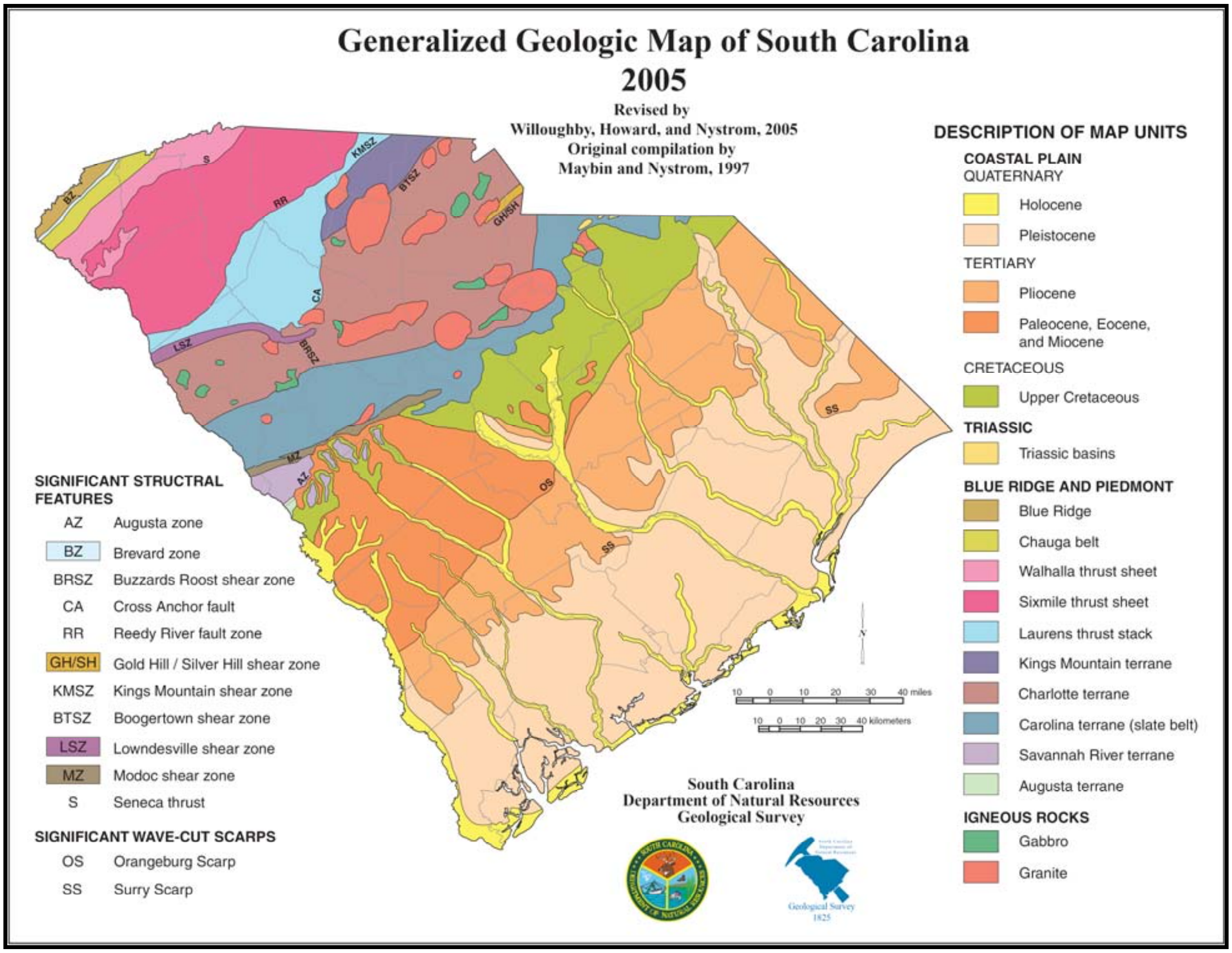


Figure 11-2, 2005 Generalized Geologic Map of South Carolina, (SCDNR)

A description of the geologic formations, age, and geologic features for the Blue Ridge, Piedmont, and Coastal Plain Physiographic Units are provided in the following sections.

Geologic Time Scale for South Carolina

(not scaled for geologic time or thickness of deposits)

EON	ERA	PERIOD		EPOCH	Geologic Events in South Carolina	MYA*
PHANEROZOIC	CENOZOIC	QUATERNARY		HOLOCENE	Barrier Islands formed; flood plains of major rivers established.	0.01
PHANEROZOIC	CENOZOIC	QUATERNARY		PLEISTOCENE	Surficial deposits cover the underlying Coastal Plain formations. Carolina Bays develop; scarps form due to sea level rise and fall.	1.6
PHANEROZOIC	CENOZOIC	TERTIARY	NEOGENE	PLIOCENE	Coastal Plain sediments reflect large-scale regressive cycles. Off-lap of the ocean and scouring responsible for the Orangeburg scarp.	5.3
PHANEROZOIC	CENOZOIC	TERTIARY	NEOGENE	MIOCENE	Uplift and erosion of Piedmont and mountains. Fluvial sediments spread over the Coastal Plain. Sandhill dunes deposited.	23
PHANEROZOIC	CENOZOIC	TERTIARY	PALEOGENE	OLIGOCENE	Deposition of carbonates predominate. Arches and embayments continue to influence deposition of Coastal Plain formations.	36.6
PHANEROZOIC	CENOZOIC	TERTIARY	PALEOGENE	EOCENE	Sand deposited in upper Coastal Plain; limestone deposited in middle and lower Coastal Plain. Fault activity.	53
PHANEROZOIC	CENOZOIC	TERTIARY	PALEOGENE	PALEOCENE	Fluvial, marginal marine and marine Coastal Plain sediments deposited.	65
PHANEROZOIC	MESOZOIC	CRETACEOUS			Development of the Cape Fear Arch and South Georgia Embayment influences deposition of Coastal Plain formations. Fault activity.	135
PHANEROZOIC	MESOZOIC	JURASSIC			Renewed sea floor spreading; intrusion of N-S and NW-SE trending diabase (basaltic) dikes. Great North American intrusive event.	205
PHANEROZOIC	MESOZOIC	TRIASSIC			Breakup of the supercontinent Pangea. Triassic rift-basins develop and fault activity.	250
PHANEROZOIC	PALEOZOIC	PERMIAN			Alleghanian Orogeny - closing of the Iapetus Ocean accompanied by continental collision and formation of the supercontinent Pangea. Rocks related to South Carolina are folded and thrusted; some rocks may have been metamorphosed.	290
PHANEROZOIC	PALEOZOIC	PENNSYLVANIAN		Emplacement of igneous intrusions	Time of uplift and erosion.	320
PHANEROZOIC	PALEOZOIC	MISSISSIPPIAN		Emplacement of igneous intrusions	Time of uplift and erosion.	355
PHANEROZOIC	PALEOZOIC	DEVONIAN		Emplacement of igneous intrusions	Arcadian Orogeny - rocks related to South Carolina may have been folded, faulted, and metamorphosed.	408
PHANEROZOIC	PALEOZOIC	SILURIAN		Emplacement of igneous intrusions	Laurentia and western South America/Africa shear apart as the Gondwanian supercontinent breakup begins.	438
PHANEROZOIC	PALEOZOIC	ORDOVICIAN		Emplacement of igneous intrusions	Taconic Orogeny - collision of Laurentia with western South America/Africa; Gondwanian supercontinent forms. Rocks related to South Carolina are folded, sheared/faulted, and metamorphosed.	510
PHANEROZOIC	PALEOZOIC	CAMBRIAN		Emplacement of igneous intrusions	Deposition of volcanic and sedimentary rocks found in the Slate belt.	570
PRECAMBRIAN	PROTEROZOIC EON				Opening of the Iapetus Ocean (750 to 700 million years ago) and continental rifting of Laurentia's (North America) eastern margin.	2,500
PRECAMBRIAN	ARCHEAN EON				Grenville Orogeny (1,100 million years ago) metamorphosed basement rocks and rocks related to the Blue Ridge. Oldest rock dated in South Carolina is 1,200 million years old.	3,800
						4,600

* Estimated age in millions of years.

MYA = million years ago

Oldest known rock in U.S. - 3,600 million years old. Oldest known rock in world - 3,850 million years old. Formation of the Earth - 4,600 million years old.

Based on the 1989 Global Stratigraphic Chart, International Union of Geological Sciences.

Figure 11-3, Geologic Time Scale for South Carolina (SCDNR)

11.3 BLUE RIDGE UNIT

The Blue Ridge Unit consists of mountains that are part of the Blue Ridge Mountains and is a southern continuation of the Appalachian Mountains. The Brevard Fault zone (depicted as the Brevard zone, BZ, in Figure 11-2) separates the Blue Ridge Unit from the Piedmont Unit. It consists of metamorphic and igneous rocks. The topography is rugged and mountainous and contains the highest elevations in the State of South Carolina with elevations ranging from 1,400 feet to 3,500 feet. Sassafras Mountain is the highest point in South Carolina with an elevation of 3,560 feet. The Appalachian Mountains were formed in the late Paleozoic era, about 342 million years ago (MYA). The basement rocks in the Blue Ridge Unit were formed in the late Precambrian time period (570 to 2,500 MYA). The oldest rock dated in South Carolina is 1,200 million years old.

The bedrock in this region is a complex crystalline formation that has been faulted and contorted by past tectonic movements. The rock has weathered to residual soils that form the mantle for the hillsides and hilltops. The typical residual soil profile in areas not disturbed by erosion or the activities of man consists of clayey soils near the surface where weathering is more advanced, underlain by sandy silts and silty sands. There may be colluvial (old land-slide) material on the slopes.

11.4 PIEDMONT UNIT

The Piedmont Unit is bounded on the west by the Blue Ridge Unit and on the east by the Coastal Plain Unit. The boundary between the Blue Ridge Unit and the Piedmont Unit is typically assumed to be the Brevard Fault zone (depicted as the Brevard zone, BZ, in Figure 11-2). The common boundary between the Piedmont Unit and the Coastal Plain Unit is the “Fall Line”. It is believed that the Piedmont is the remains of an ancient mountain chain that has been eroded with existing elevations ranging from 300 feet to 1,400 feet. The Piedmont is characterized by gently rolling topography, deeply weathered bedrock, and relatively few rock outcrops. It contains monadnocks that are isolated outcrops of bedrock (usually quartzite or granite) that are a result of the erosion of the mountains. The vertical stratigraphic sequence consists of 5 to 70 feet of weathered residual soils at the surface underlain by metamorphic and igneous basement rocks (granite, schist, and gneiss). The weathered soils (saprolites) are physically and chemically weathered rocks that can be soft/loose to very hard and dense, or friable and typically retain the structure of the parent rock. The geology of the Piedmont is complex with numerous rock types that were formed during the Paleozoic era (250 to 570 MYA).

The typical residual soil profile consists of clayey soils near the surface, where soil weathering is more advanced, underlain by sandy silts and silty sands. The boundary between soil and rock is not sharply defined. This transitional zone termed “partially weathered rock” (PWR) is normally found overlying the parent bedrock. Partially weathered rock is defined, for engineering purposes, as residual material with Standard Penetration Test resistances in excess of 100 blows/foot. The partially weathered rock is considered in geotechnical engineering as an Intermediate Geomaterial (IGM). Weathering is facilitated by fractures, joints, and by the presence of less resistant rock types. Consequently, the profile of the partially weathered rock and hard rock is quite irregular and erratic, even over short horizontal distances.

Also, it is not unusual to find lenses and boulders of hard rock and zones of partially weathered rock within the soil mantle, well above the general bedrock level.

11.5 "FALL LINE"

A "Fall Line" is an unconformity that marks the boundary between an upland region (bed rock) and a coastal plain region (sediment). In South Carolina the Piedmont Unit is separated from the Coastal Plain Unit by a "Fall Line" that begins near the Edgefield-Aiken County line and traverses to the northeast through Lancaster County. In addition to Columbia, SC many cities were built along the "Fall Line" as it runs up the east coast (Macon, Raleigh, Richmond, Washington D.C., and Philadelphia). The "Fall Line" generally follows the southeastern border of the Savannah River terrane formation and the Carolina terrane (slate belt) formation shown in Figure 11-2. Along the "Fall Line" between elevations 300 to 725, the Sandhills formations can be found which are the remnants of a prehistoric coastline. The Sandhills are unconnected bands of sand deposits that are remnants of coastal dunes that were formed during the Miocene epoch (5.3 to 23 MYA). The land to the southeast of the "Fall Line" is characterized by a gently downward sloping elevation (2 to 3 feet per mile) as it approaches the Atlantic coastline as shown in Figure 11-4. Several rivers such as the Pee Dee, Wateree, Lynches, Congaree, N. Fork Edisto, and S. Fork Edisto flow from the "Fall Line" towards the Atlantic coast as they cut through the Coastal Plain sediments.

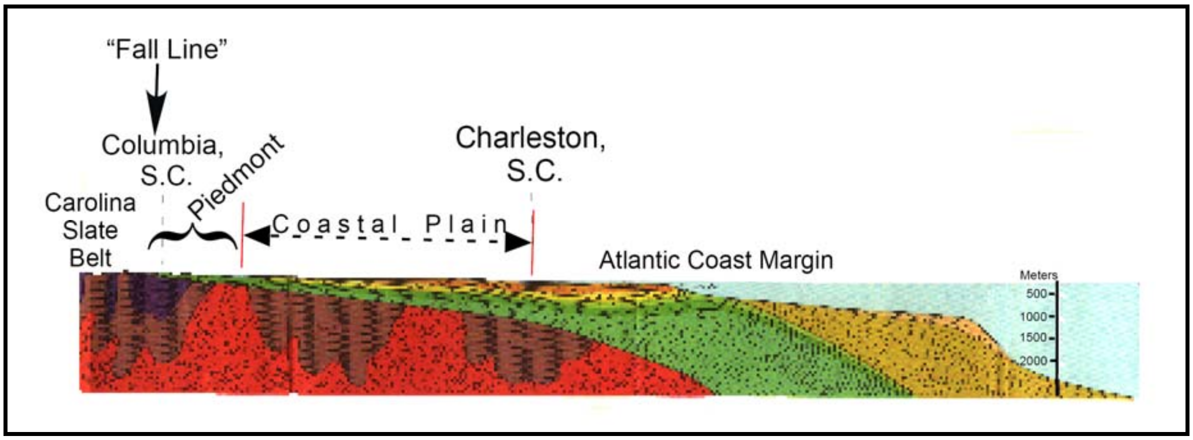


Figure 11-4, South Carolina "Fall Line"
(Odum et al., 2003)

11.6 COASTAL PLAIN UNIT

The Coastal Plain Unit is a compilation of wedge shaped formations that begin at the "Fall Line" and dip towards the Atlantic Ocean with ground surface elevations typically less than 300 feet. The Coastal Plain is underlain by Mesozoic/Paleozoic basement rock. This wedge of sediment is comprised of numerous geologic formations that range in age from late Cretaceous period to Recent. The sedimentary soils of these formations consist of unconsolidated sand, clay, gravel, marl, cemented sands, and limestone that were deposited over the basement rock. The marl and limestone are considered in geotechnical engineering as an IGM. The basement rock consists of granite, schist, and gneiss similar to the rocks of the Piedmont Unit. The thickness of the Coastal Plain sediments varies from zero at the "Fall Line" to more than 4,000 feet at the southern tip of South Carolina near Hilton Head Island. The thickness of the Coastal Plain sediments along the Atlantic coast varies from ~1300 feet at Myrtle Beach to ~4000 feet at

Hilton Head Island. The top of the basement beneath the Coastal Plain has been mapped during a SC Seismic Hazard Study that was prepared for SCDOT and the contours of the Coastal Plain sediment thickness in meters are shown in Figure 11-5.

The area is formed of older, generally well-consolidated layers of sands, silts, or clays that were deposited by marine or fluvial action during a period of retreating ocean shoreline. Predominantly, sediments lie in nearly horizontal layers; however, erosional episodes occurring between depositions of successive layers are often expressed by undulations in the contacts between the formations. Due to their age, sediments exposed at the ground surface are often heavily eroded. Ridges and hills are either capped by terrace gravels or wind-deposited sands. Younger alluvial soils may mask these sediments in swales or stream valleys.

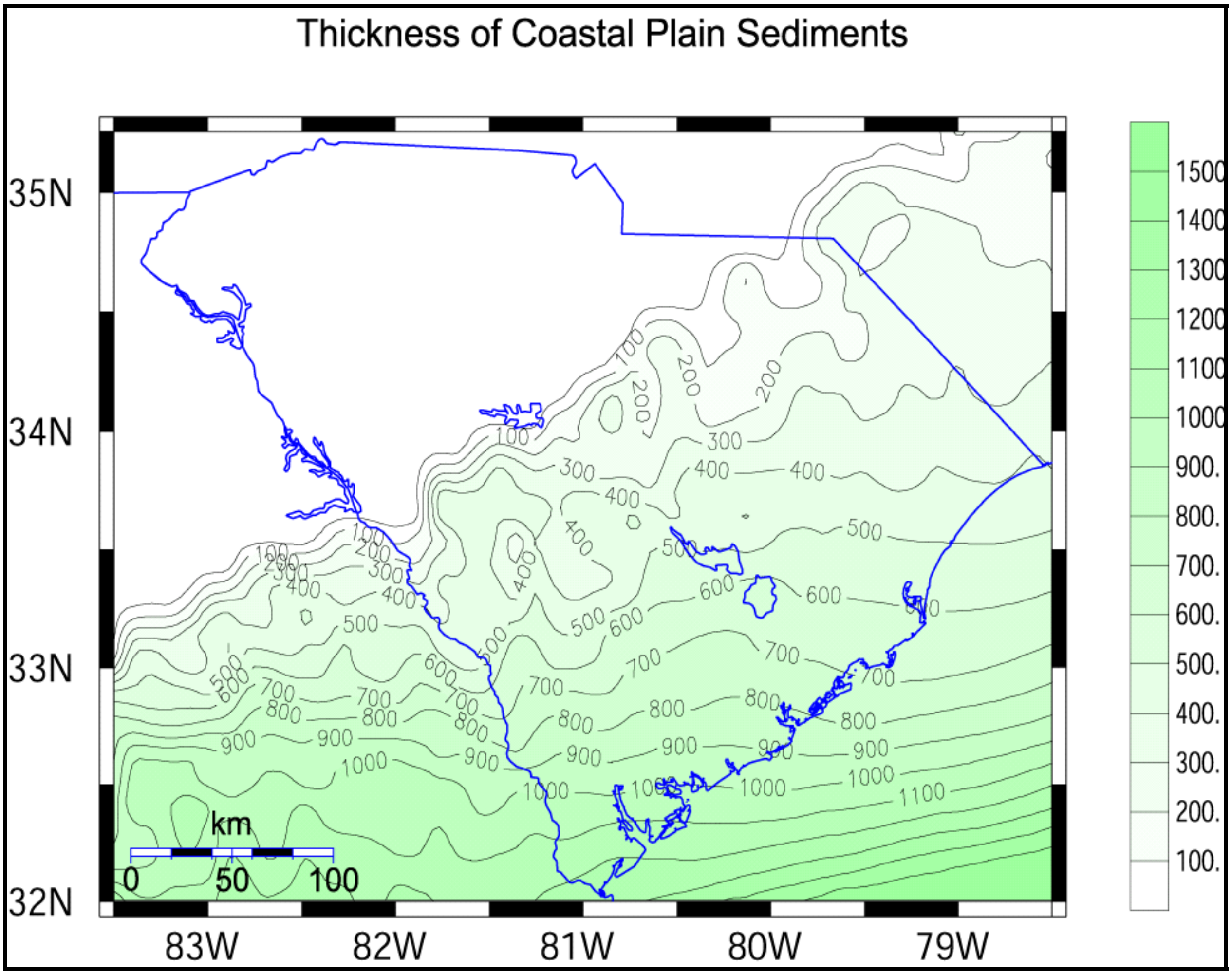


Figure 11-5, Contour Map of Coastal Plain Sediment Thickness, in meters (Chapman and Talwani, 2002)

This Coastal Plain Unit was formed during Quaternary, Tertiary, and late Cretaceous geologic periods. The Coastal Plain can be divided into the following three subunits:

- Upper Coastal Plain
- Middle Coastal Plain
- Lower Coastal Plain

The Lower Coastal Plain comprises approximately one-half of the entire Atlantic Coastal Plain of South Carolina. The Surry Scarp (-SS-) shown in Figure 11-2 separates the Lower Coastal Plain from the Middle Coastal Plain. The Surry Scarp is a seaward facing scarp with a toe elevation of 90 to 100 feet. The Middle Coastal Plain and the Upper Coastal Plain each compose approximately one fourth of the Coastal Plain area. The Orangeburg Scarp (-OS-) shown in Figure 11-2 separates the Middle Coastal Plain from the Upper Coastal Plain. The Orangeburg Scarp is also a seaward facing scarp with a toe elevation of 250 to 270 feet.

11.6.1 <u>Lower Coastal Plain</u>

The Lower Coastal Plain is typically identified as the area east of the Surry Scarp below elevation 100 feet. The vertical stratigraphic sequence overlying the basement rock consists of unconsolidated Cretaceous, Tertiary, and Quaternary sedimentary deposits. The surface deposits of the Lower Coastal Plain were formed during the Quaternary period that began approximately 1.6 MYA and extends to present day. The Quaternary period can be further subdivided into the Pleistocene epoch and the Holocene epoch. During the Pleistocene epoch (1.6 MYA to 10 thousand years ago) the surficial deposits that cover the underlying Coastal Plain formations were formed. This period specifically marks the formation of the Carolina Bays and scarps throughout the east coast due to sea level rise and fall. The Holocene epoch covers from 10 thousand years ago to present day. Barrier islands were formed and flood plains from major rivers were formed during the Holocene epoch. Preceding Quaternary period during the Eocene epoch (53 to 36.6 MYA) of the Tertiary period, limestone was deposited in the Lower Coastal Plain.

11.6.2 <u>Middle Coastal Plain</u>

The Middle Coastal Plain is typically identified as the area between the Orangeburg Scarp and the Surry Scarp and falls between elevation 100 feet and 270 feet. The vertical stratigraphic sequence overlying the basement rock consists of unconsolidated Cretaceous and Tertiary sedimentary deposits. The surface deposits of the Middle Coastal Plain were formed during the Pliocene epoch of the Tertiary period. During the Pliocene epoch (5.3 to 1.6 MYA) of the Tertiary period, the Orangeburg Scrap was formed as a result of scouring from the regressive cycles of the Ocean as it retreated. During the Eocene epoch (53 to 36.6 MYA) of the Tertiary period, limestone was deposited in the Middle Coastal Plain.

11.6.3 <u>Upper Coastal Plain</u>

The Upper Coastal Plain is typically identified as the area between the “Fall Line” and the Orangeburg Scarp and falls between elevations 270 feet and 300 feet. The Upper Coastal Plain was formed during the Tertiary and late Cretaceous periods. The Tertiary period began approximately 65 MYA and ended approximately 1.6 MYA. The Tertiary period can be further subdivided into the Pliocene epoch, Miocene epoch, Oligocene epoch, Eocene epoch, and Paleocene epoch. The Miocene epoch (23 to 5.3 MYA) is marked by the formation of the Sandhills dunes as a result of fluvial deposits over the Coastal Plain. During the early Tertiary period (65 to 23 MYA) fluvial deposits over the Coastal Plain consisted of marine sediments, limestone, and sand.

11.7 SOUTH CAROLINA SEISMICITY

11.7.1 Central and Eastern United States (CEUS) Seismicity

Even though seismically active areas in the United States are generally considered to be in California and Western United States, historical records indicate that there have been major earthquake events in Central and Eastern United States (CEUS) that have not only been of equal or greater magnitude but that have occurred over broader areas of the CEUS. The United States Geologic Survey (USGS) map shown in Figure 11-6 indicates earthquakes that have caused damage within the United States between 1750 and 1996. Of particular interest to South Carolina is the 1886 earthquake in Charleston, SC that has been estimated to have a M_W of at least 7.3. Also of interest to the northwestern end of South Carolina is the influence of New Madrid seismic zone, near New Madrid, Missouri, where historical records indicate that between 1811 and 1812 there were several large earthquakes with a M_W of at least 7.7.

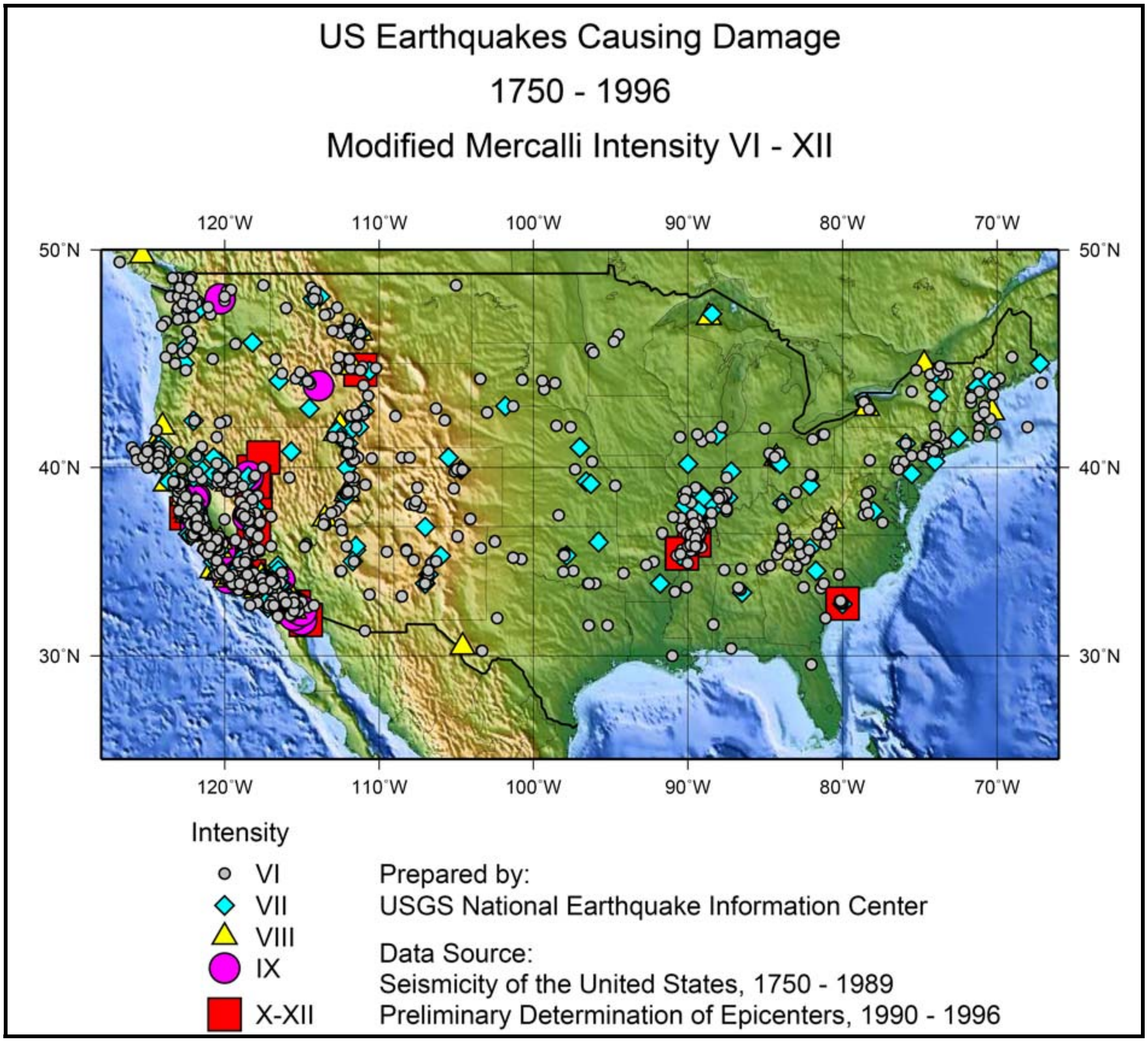


Figure 11-6, U.S. Earthquakes Causing Damage 1750 – 1996 (USGS)

11.7.2 SC Earthquake Intensity

The Modified Mercalli Intensity Scale (MMIS) is a qualitative measure of the strength of ground shaking at a particular site that is used in the United States. Each earthquake large enough to be felt will have a range of intensities. Typically the highest intensities are measured near the earthquake epicenter and lower intensities are measured farther away. The MMIS is used to distinguish the ground shaking at geographic locations as opposed to the moment magnitude scale that is used to compare the energy released by earthquakes. Roman numerals are used to identify the MMIS of ground shaking with respect to shaking and damage felt at a geographic location as shown in Table 11-1.

Table 11-1, Modified Mercalli Intensity Scale (MMIS)

INTENSITY	I	II – III	IV	V	VI	VII	VIII	IX	X+
SHAKING	Not Felt	Weak	Light	Moderate	Strong	Very Strong	Severe	Violent	Extreme
DAMAGE	None	None	None	Very Light	Light	Moderate	Moderate / Heavy	Heavy	Very Heavy

Figure 11-7 shows a map developed by the South Carolina Geological Survey with earthquake intensities, by county, based on the MMIS. The intensities shown on this map are the highest likely under the most adverse geologic conditions that would be produced by a combination of the August 31, 1886, Charleston, S.C. earthquake (M_W = 7.3) and the January 1, 1913, Union County, S.C., earthquake (M_W = 5.5). This map is for informational purposes only and is not intended as a design tool, but reflects the potential for damage based on earthquakes similar to the Union and Charleston earthquake events.

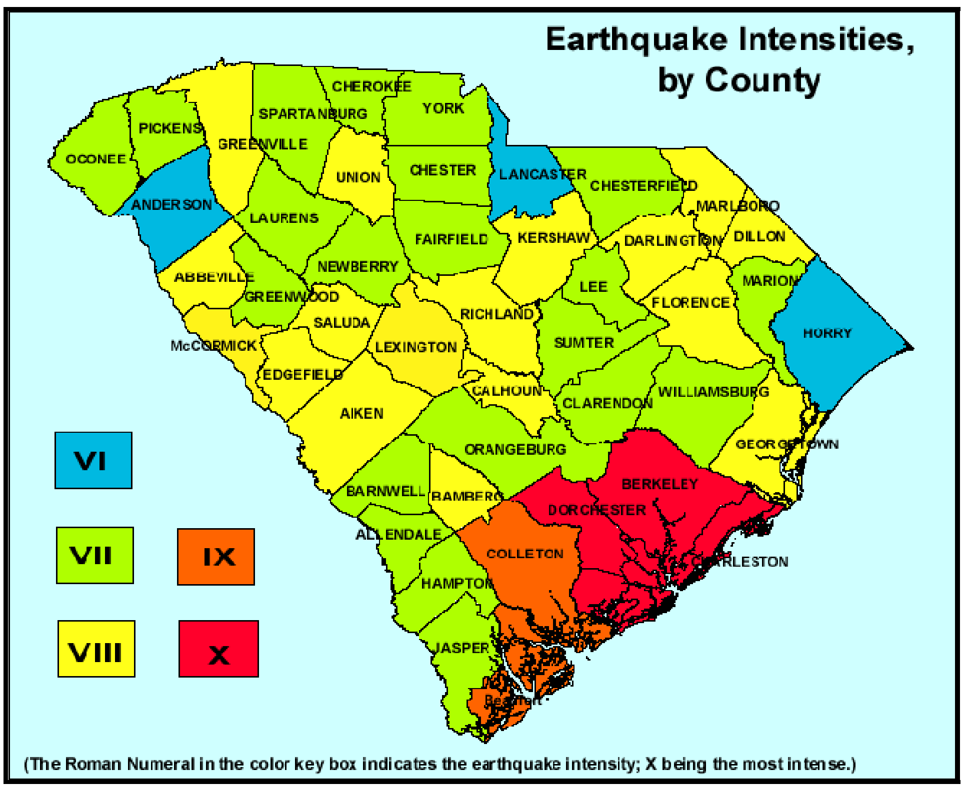


Figure 11-7, SC Earthquake Intensities By County (SCDNR)

11.8 SOUTH CAROLINA SEISMIC SOURCES

Sources of seismicity are not well defined in much of the Eastern United States. South Carolina seismic sources have therefore been defined based on seismic history in the Southeastern United States. The SC Seismic Hazard study (Chapman and Talwani, 2002) has identified two types of seismic sources: Non-Characteristic Earthquakes and Characteristic Earthquakes.

11.8.1 Non-Characteristic Earthquake Sources

Seismic histories were used to establish seismic area sources for analysis of non-characteristic background events. The study modified the Frankel et al., 1996 source area study to develop the seismic source areas shown in Figures 11-8 and 11-9.

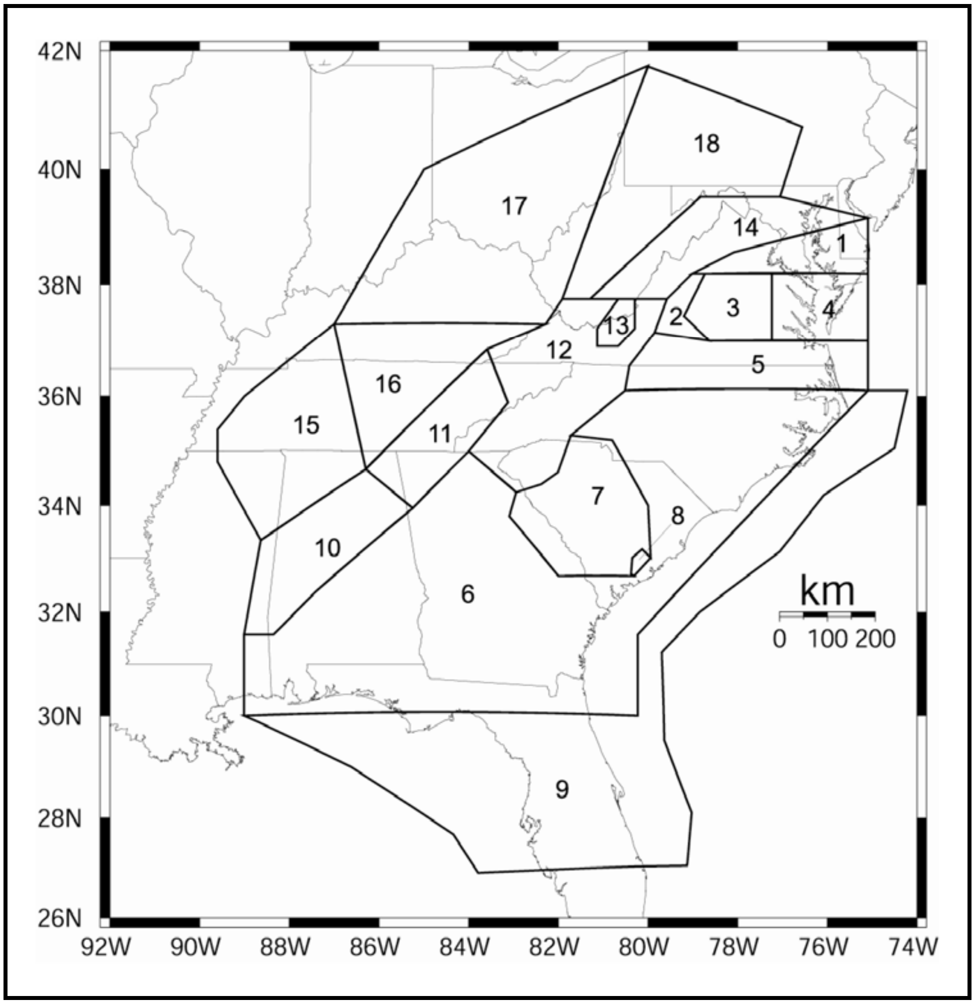


Figure 11-8, Source Areas for Non-Characteristic Earthquakes (Chapman and Talwani, 2002)

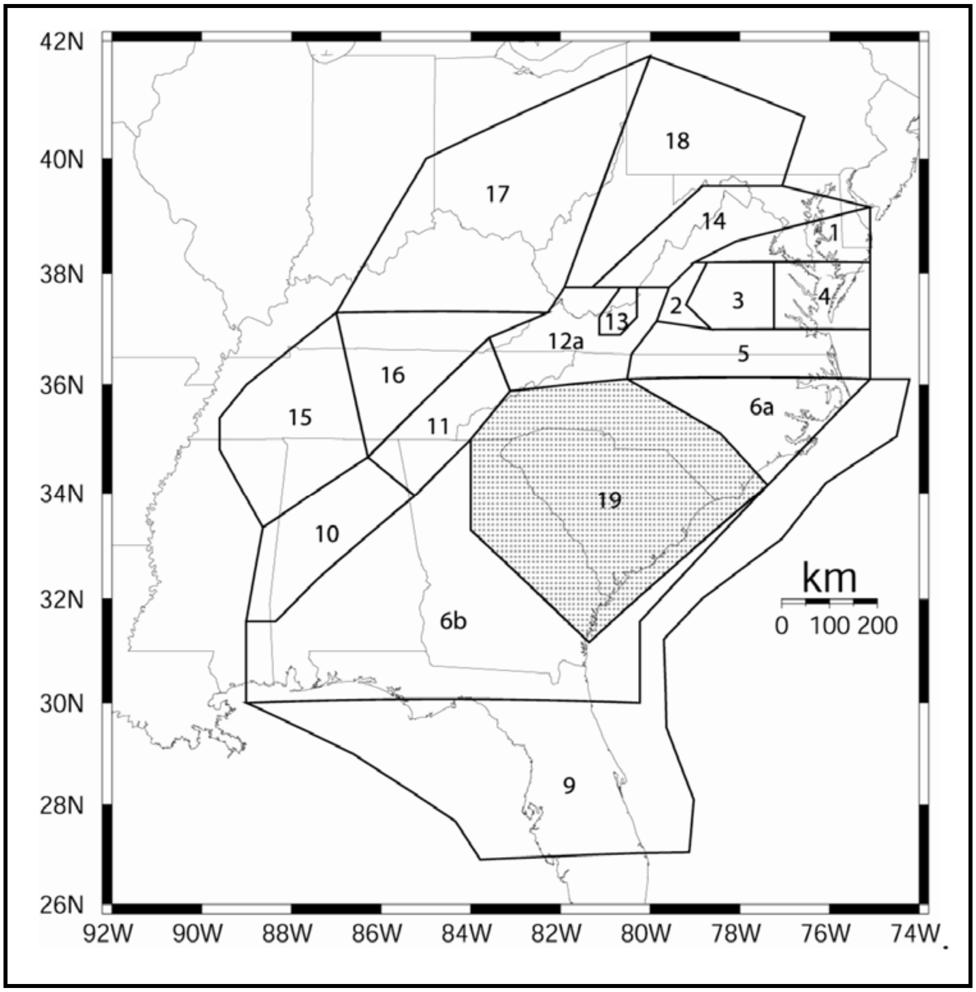


Figure 11-9, Alternative Source Areas for Non-Characteristic Earthquakes (Chapman and Talwani, 2002)

The source areas listed in Figures 11-8 and 11-9 are described in Table 11-2.

Table 11-2, Source Areas for Non-Characteristic Background Events (Chapman and Talwani, 2002)

Area No.	Description	Area (sq.miles)	Area No.	Description	Area (sq.miles)
1	Zone 1	8,133	**10**	Alabama	20,257
2	Zone 2	2,475	**11**	Eastern Tennessee	14,419
3	Central Virginia	7,713	**12**	Southern Appalachian	29,234
4	Zone 4	9,687	**12a**	Southern Appalachian N.	17,034
5	Zone 5	18,350	**13**	Giles County, VA	1,980
6	Piedmont and Coastal Plain	161,110	**14**	Central Appalachians	16,678
6a	Piedmont & CP NE	18,815	**15**	West Tennessee	29,667
6b	Piedmont & CP SW	95,854	**16**	Central Tennessee	20,630
7	SC Piedmont	22,248	**17**	Ohio – Kentucky	58,485
8	Middleton Place	455	**18**	West VA-Pennsylvania	34,049
9	Florida/Continental Margin	110,370	**19**	USGS Gridded Seis.-1996	---

Figure 11-10 shows additional historical seismic information obtained from the Virginia Tech catalog of seismicity in the Southeastern United States from 1600 to present that was used to model the non-characteristic background events in the source areas.

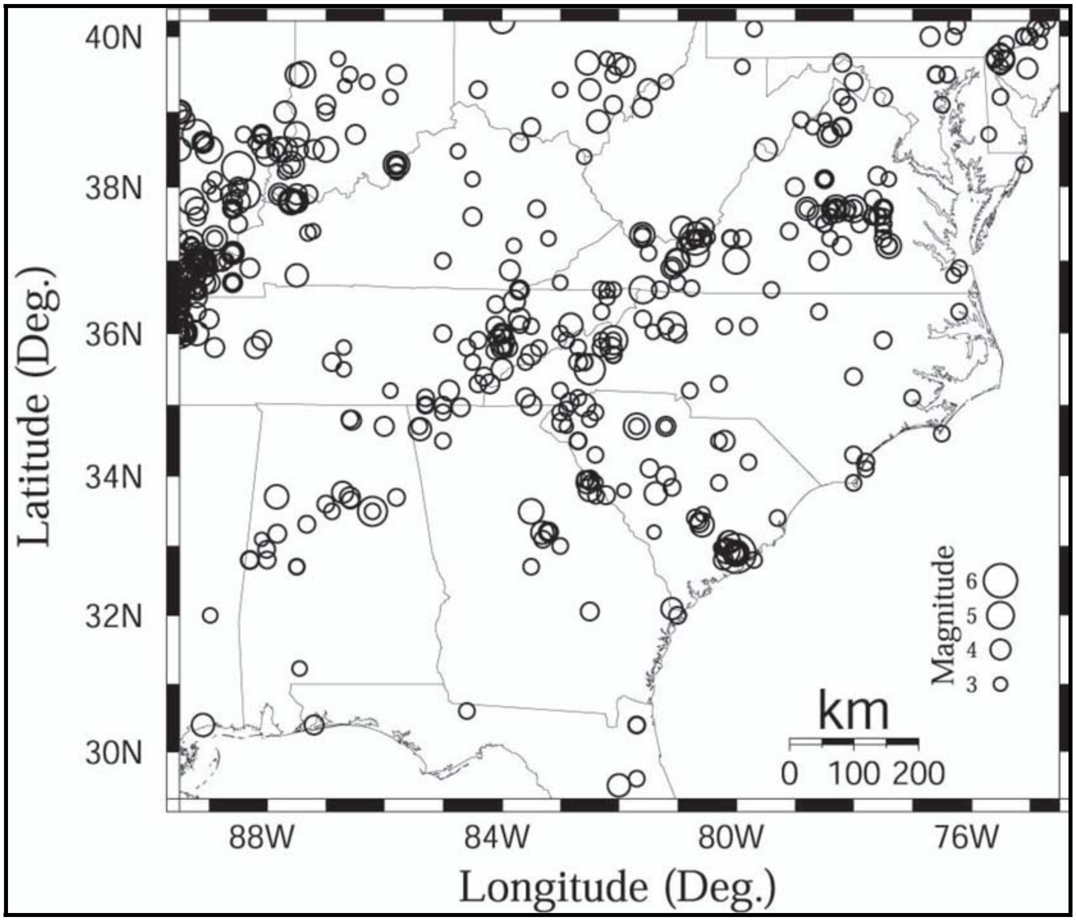


Figure 11-10, Southeastern U.S. Earthquakes (M_W > 3.0 from 1600 to Present) (Chapman and Talwani, 2002)

11.8.2 Characteristic Earthquake Sources

The single most severe earthquake that has occurred in South Carolina's human history occurred in Charleston, South Carolina, in 1886. It was one of the largest, earthquakes to affect the Eastern United States in historical times. The M_W of this earthquake has been estimated to range from 7.0 to 7.5. It is typically referred to have a M_W of 7.3. The faulting source that was responsible for the 1886 Charleston earthquake remains uncertain to date.

Large magnitude earthquake events with the potential to occur in coastal South Carolina are considered characteristic earthquakes. These earthquakes are modeled as a combination of fault sources and a seismic Area Source. The SC Seismic Hazard study used the 1886 Earthquake fault source, also known as the Middleton Place seismic zone, and the "Zone of River Anomalies" (ZRA) fault source. For the 1886 Earthquake fault source it assumed that rupture occurred on the NE trending "Woodstock" fault and on the NW trending "Ashley River" fault. The 1886 Earthquake fault source is modeled as three independent parallel faults.

Recent studies (Marple and Talwani, 1993, 2000) suggest that the “Woodstock” fault may be a part of larger NE trending fault system that extends to North Carolina and possibly Virginia, referred to in the literature as the “East Coast Fault System”. The ZRA fault source is the term used for the portion of the “East Coast Fault System” that is located within South Carolina. The ZRA fault system is modeled by a 145-mile long fault with a NE trend. The characteristic seismic Area Source is the same as is used in the 1996 National Seismic Hazard Maps. It models a network of individual faults no greater than 46 miles in length within the Lower Coastal Plain. The fault sources and area sources used to model the characteristic earthquake sources in the SC Seismic Hazard Study are shown in Figure 11-11.

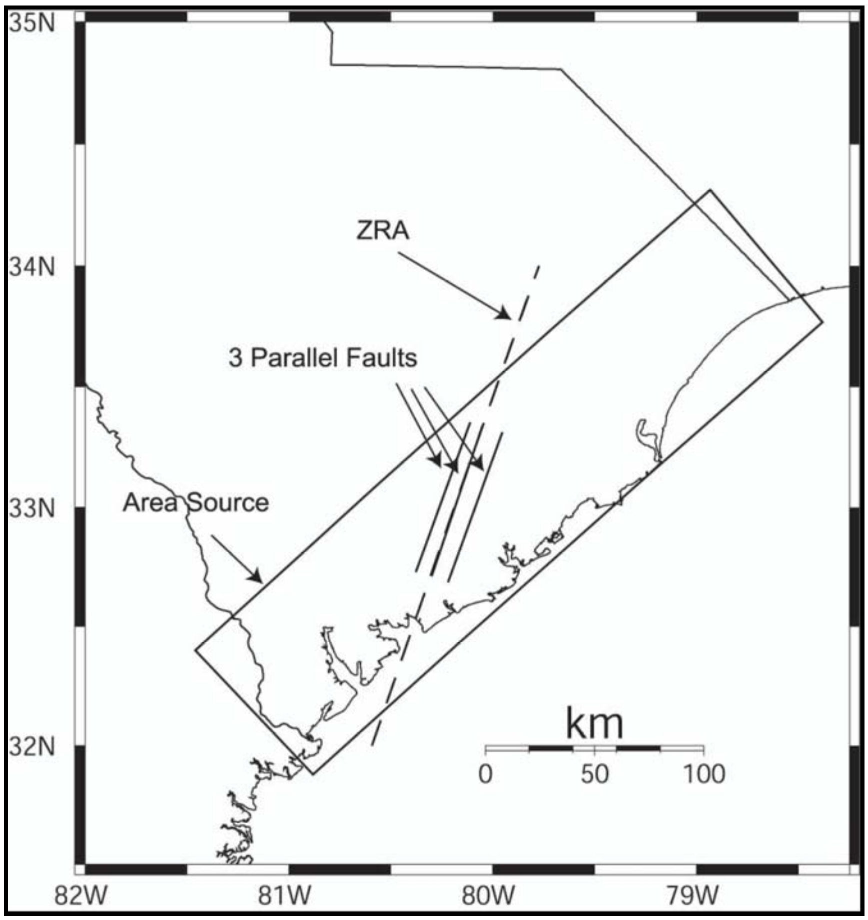


Figure 11-11, South Carolina Characteristic Earthquake Sources (Chapman and Talwani, 2002)

11.9 SOUTH CAROLINA EARTHQUAKE HAZARD

11.9.1 Design Earthquakes

The SCDOT uses a Functional Evaluation Earthquake (FEE) and a Safety Evaluation Earthquake (SEE) to design transportation infrastructure in South Carolina. The FEE represents a small ground motion that has a likely probability of occurrence within the life of the structure being designed. The SEE represents a large ground motion that has a relatively low probability of occurrence within the life of the structure. The two levels of earthquakes have been chosen for South Carolina because SEE spectral accelerations can be as much as three to four times higher than FEE spectral accelerations in the Eastern United States. In contrast, the California SEE spectral accelerations can be the same or as much as 1.8 times the FEE spectral accelerations. Because of the large variation between FEE and SEE design earthquake events it is necessary to perform geotechnical earthquake engineering analyses for each event and compare the resulting performance with the SCDOT Performance Limits established in Chapter 10. The design life for transportation infrastructure is typically assumed to be 75 years when evaluating the design earthquakes, regardless of the actual design life specified in Chapter 10. The likelihood of these events occurring is quantified by the design events probability of exceedance (P_E) within the design life of the structure. Descriptions of the design earthquakes used in South Carolina are provided in Table 11-3.

Table 11-3, SCDOT Design Earthquakes

Design Earthquake	Description
Functional **E**valuation **E**arthquake (**FEE**)	The ground shaking having a 15 percent probability of exceedance in 75 years (15%/75 year). This design earthquake is equal to the 10 percent probability of exceedance in 50 years (10%/50). The FEE PGA and PSA are used for the functional evaluation of transportation infrastructure.
Safety **E**valuation **E**arthquake (**SEE**)	The ground shaking having a 3 percent probability of exceedance in 75 years (3%/75 year). This design earthquake is equal to the 2 percent probability of exceedance in 50 years (2%/50). The SEE PGA and PSA are used for the safety evaluation of transportation infrastructure.

11.9.2 Probabilistic Earthquake Hazard Maps

A SC Earthquake Hazard study was completed for SCDOT In October 2006 (Chapman and Talwani, 2002 and Chapman, 2006). The study produced probabilistic seismic hazard maps that reflect the actual geological conditions in South Carolina. The seismic hazard maps are motion intensities for a specific probability of exceedance (P_E). The motions are defined in terms of pseudo-spectral accelerations (*PSA*) at frequencies of 0.5, 1.0, 2.0, 3.3, 5.0, 6.67, and 13.0 Hz, for a damping ratio of 0.05 (5%) and the peak horizontal ground acceleration (PHGA or PGA). These accelerations were developed for the geologically realistic site conditions as well as for the hypothetical hard-rock basement outcrop. The geologically realistic site condition is a hypothetical site condition that was developed by using a transfer function of a linear response. South Carolina has been divided into two zones as shown in Figure 11-12: Zone I – Physiographic Units Outside of the Coastal Plain and Zone II – Coastal Plain Physiographic Unit. The delineation between these two zones has been shown linearly in Figure 11-12 but in reality it should follow the “Fall Line.” Because of the distinct differences between these two physiographic units, a geologically realistic model has been developed for each zone.

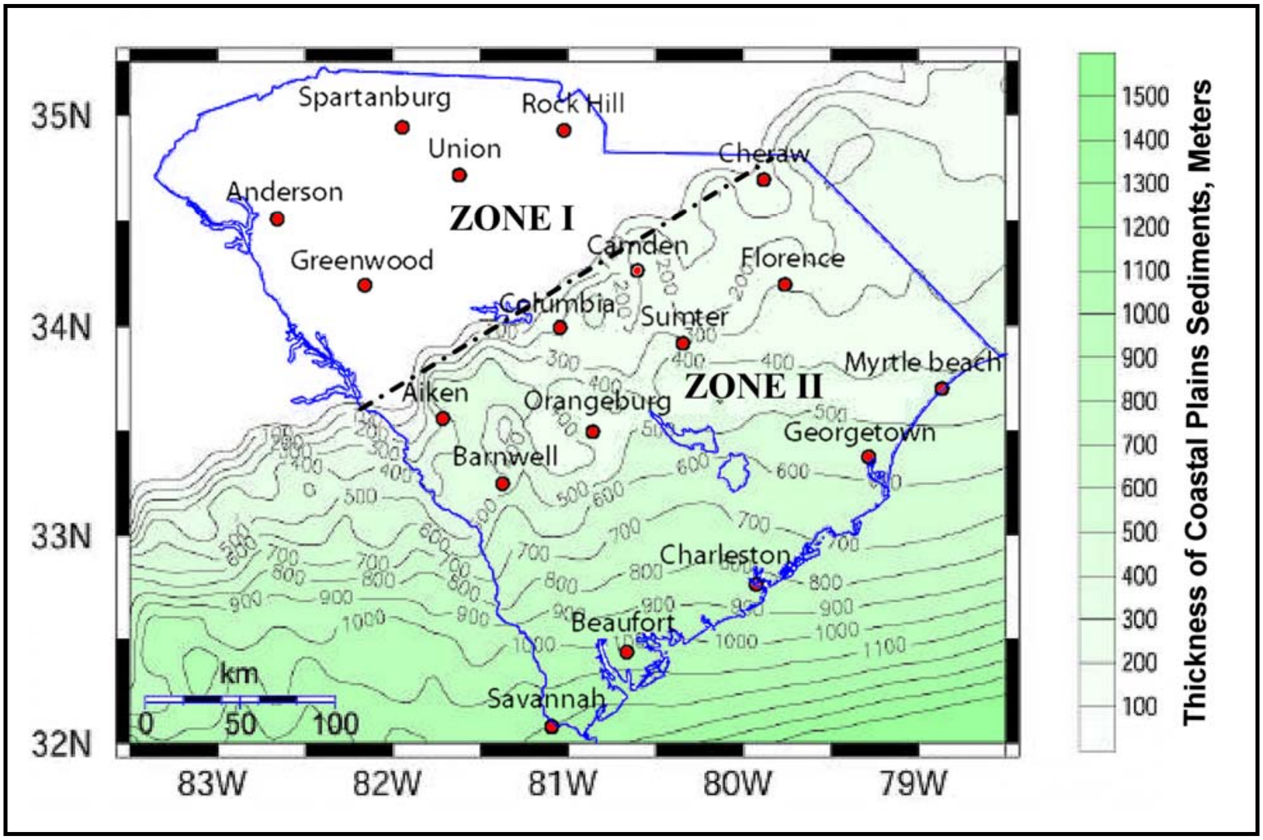


Figure 11-12, SCDOT Site Condition Selection Map (Modified Chapman and Talwani, 2002)

The Coastal Plain geologically realistic site condition consists of two layers, the shallowest layer consists of Coastal Plain sedimentary soil (Q=100) and weathered rock (Q=600), over a half-space of unweathered Mesozoic and Paleozoic sedimentary, and Metamorphic/Igneous rock, assuming vertical shear wave incidence. The soil properties for the Coastal Plain geologically realistic model are shown in Table 11-4.

The Piedmont geologically realistic site condition consists of one layer of weathered rock (Q=600) over a half-space of unweathered Mesozoic and Paleozoic sedimentary, and Metamorphic/Igneous rock, assuming vertical shear wave incidence. The soil properties for the Piedmont geologically realistic model are shown in Table 11-5.

Table 11-4, Coastal Plain Geologically Realistic Model

Soil Layer	Mass Density, ρ	Total Unit Weight, γ	Shear Wave Velocity, V_S
	kg/m^3	pcf	ft/sec
Layer 1 – Sedimentary Soils	2,000	125	2,300
Layer 2 – Weathered Rock	2,500	155	8,200
Half-Space – Basement Rock	2,600	165	11,200

Table 11-5, Geologically Realistic Model Outside of Coastal Plain

Soil Layer	Mass Density, ρ	Total Unit Weight, γ	Shear Wave Velocity, V_S
	kg/m^3	pcf	ft/sec
Layer 1 – Weathered Rock	2,500	155	8,200
Half-Space - Basement Rock	2,600	165	11,200

The transfer functions were computed using ¼ wavelength approximation of Boor and Joyner (1991). For more information on the development of the transfer function refer to Chapman and Talwani (2002).

The selection of the appropriate site condition is very important in the generation of probabilistic seismic hazard motions in the form of pseudo-spectral accelerations (*PSA*) and the peak horizontal ground acceleration (PHGA or PGA). The available site conditions for use in generating probabilistic seismic hazard motions are defined in Table 11-6. The selection of the appropriate site condition should be based on the results of the geotechnical site investigation, geologic maps, and any available geologic or geotechnical information from past projects in the area. Generally speaking the geologically realistic site condition should be used in the Coastal Plain. In areas outside of the Coastal Plain such as the Piedmont / Blue Ridge Physiographic Units and along the "Fall Line" should be evaluated carefully. The geotechnical investigation in these areas should be sufficiently detailed to determine depth to weathered rock having a shear wave velocity of approximately 8,000 to 8,200 ft/sec or to define the basement rock outcrop having a shear wave greater than 11,000 ft/sec.

Table 11-6, Site Conditions

<table>
<tr><th rowspan="2">South Carolina Zones</th><th colspan="2">Site Condition</th></tr>
<tr><th>Geologically Realistic</th><th>Hard-Rock Basement Outcrop</th></tr>
<tr><td>Zone I – Physiographic Units Outside of the Coastal Plain</td><td>Hypothetical outcrop of "Weathered Southeastern U.S. Piedmont Rock" that consist of 820 feet thick weathered formation of shear wave velocity, V_s = 8,000 ft/s overlying a hard-rock formation having shear wave velocity, V_s = 11,500 ft/s.</td><td rowspan="2">A hard-rock basement outcrop formation having shear wave velocity, V_s = 11,500 ft/s.</td></tr>
<tr><td>Zone II – Coastal Plain Physiographic Unit</td><td>Hypothetical outcrop of "Firm Coastal Plain Sediment" equivalent to the B-C Boundary having a shear wave velocity, V_s = 2,500 ft/s.</td></tr>
</table>

The seismic hazards computations use the seismic sources listed in Section 11.8, the design earthquake in Section 11.9.1, and the ground motions described in Section 11.9.4.

The PGA and PSA can be obtained for any location in South Carolina by specifying a Latitude and Longitude. The Latitude and Longitude of a project site may be obtained from the plans or by using an Interactive Internet search tool. Typical Latitude and Longitude for South Carolina cities are provided in Table 11-7 for reference.

Table 11-7, Latitude and Longitude for South Carolina Cities

SC City	Latitude	Longitude	SC City	Latitude	Longitude
Anderson, SC	34.50	82.72	Greenwood, SC	34.17	82.12
Beaufort, SC	32.48	80.72	Myrtle Beach, SC	33.68	78.93
Charleston, SC	32.90	80.03	Nth Myrtle B, SC	33.82	78.72
Columbia, SC	33.95	81.12	Orangeburg, SC	33.50	80.87
Florence, SC	34.18	79.72	Rock Hill, SC	34.98	80.97
Georgetown, SC	33.83	79.28	Spartanburg, SC	34.92	81.96
Greenville, SC	34.90	82.22	Sumter, SC	33.97	80.47

The site-specific hazard PGA and PSA are generated by the GDS for every project using Scenario_PC (2006) (Chapman, 2006). Scenario_PC generates seismic hazard data in a similar format as that generated by the USGS. The designer must obtain a SC Seismic Hazard request form and submit it to the GDS. A copy of the form is included in Appendix A. The SC Seismic Hazard request form requires that the designer provide the following information.

- SCDOT Project Name and Project Number
- Latitude and Longitude of Project Site
- Probability of Exceedance for Earthquake Design Event being analyzed
- Site Condition: Geologically Realistic or Hard-Rock Basement Outcrop

The geotechnical engineer is required to provide documentation for the selection of the Site Condition (Geologically Realistic or Hard-Rock Basement Outcrop) used.

A sample of the Seismic Hazard information generated by Scenario_PC (2006) for Columbia, SC is shown in Figure 11-13.

```
THE NAME OF THE DIRECTORY CONTAINING THIS FILE
 AND ALL ASSOCIATED OUTPUT FILES IS: Columbia


 3% PROBABILITY OF EXCEEDANCE (For 75 year Exposure)
      FOR GEOLOGICALLY REALISTIC SITE CONDITION


 RESULTS OF INTERPOLATION

     Site Location: 33.9500 N 81.1200 W
Nearest Grid Point: 34.0000 N 81.1250 W Distance From Site:   5.56 Km
Thickness of sediments, meters: 262.162

                    PSA and PGA as Percentage of g
  0.5Hz     1.0Hz     2.0Hz     3.3Hz     5Hz       6.7Hz     13Hz      PGA
 6.36404  18.97654  30.64109  40.70470  46.59745  45.10500  40.47712  19.61478
```

Figure 11-13, Scenario_PC (2006) Sample Output for Columbia, SC

In order to provide the designer with an overview of the South Carolina's probabilistic seismic hazard, probabilistic seismic hazard contour maps for the FEE and SEE design events for PGA, PSA for the short-period, S_s, (5 Hz = 0.2 seconds), and PSA for the long-period, S_1, (1 Hz = 1.0 second) have been included in this Chapter. The PGA and PSA values as a percentage of gravity (g) have been placed in contours and overlaid over a South Carolina map. FEE seismic hazard contour maps are provided for PGA, S_s, and S_1 in Figures 11-14, 11-15, and 11-16, respectively. SEE seismic hazard contour maps are provided for PGA, S_s, and S_1 in Figures 11-17 11-18, and 11-19, respectively. FEE and SEE peak ground accelerations (PGA) and pseudo-spectral accelerations (PSA) (generated by Scenario_PC 2006) for selected cities in South Carolina have been plotted at either the B-C boundary (geologically realistic) or hard rock basement outcrop in Figures 11-20 and 11-21. <u>The seismic hazard contour maps and the sampling of the PSA curves for various cities are provided for information only and must not be used for design of any structures in South Carolina.</u>

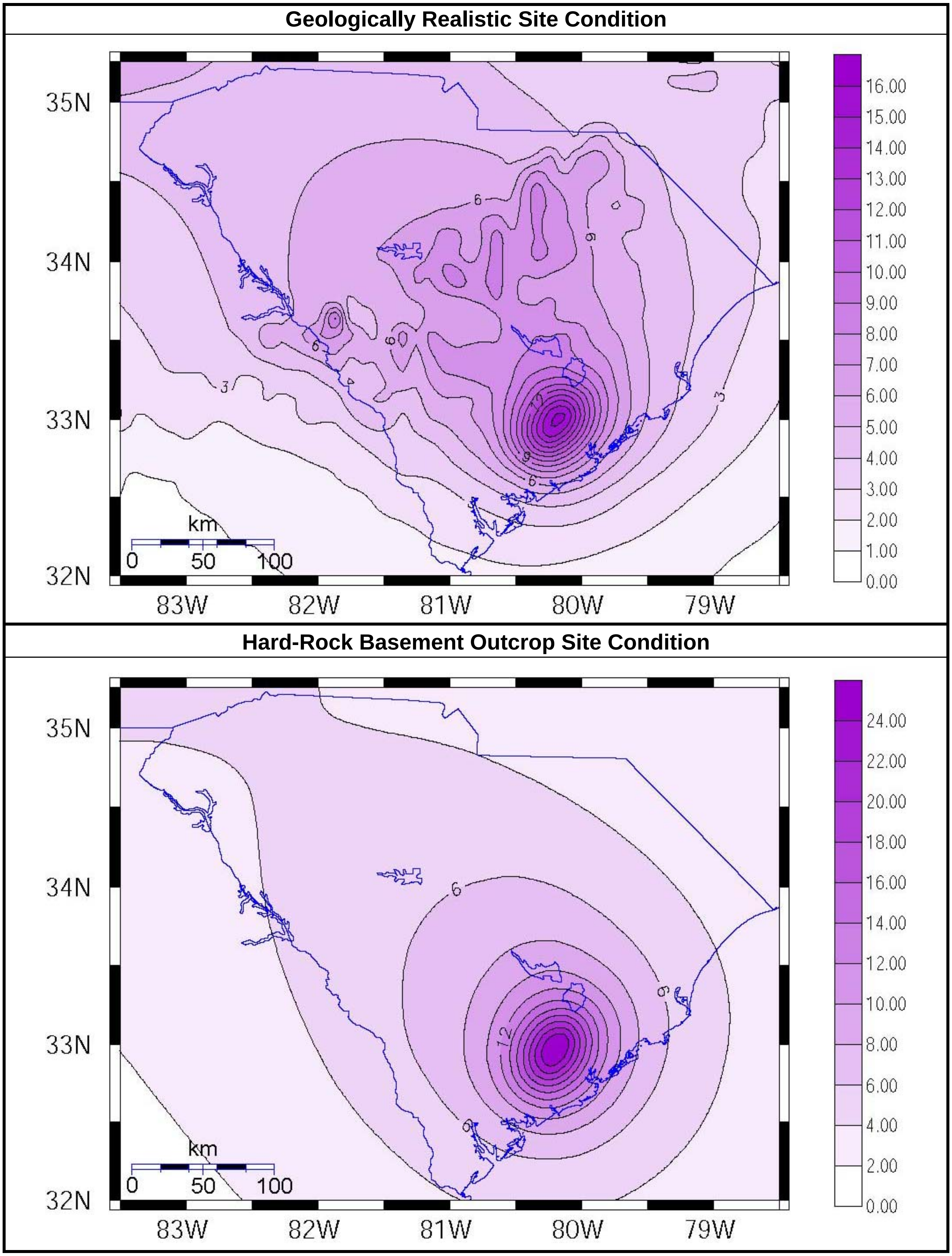


Figure 11-14, PGA (%g) - 15% P_E in 75 Years (10% P_E-50 Yr)
(Chapman and Talwani, 2002)

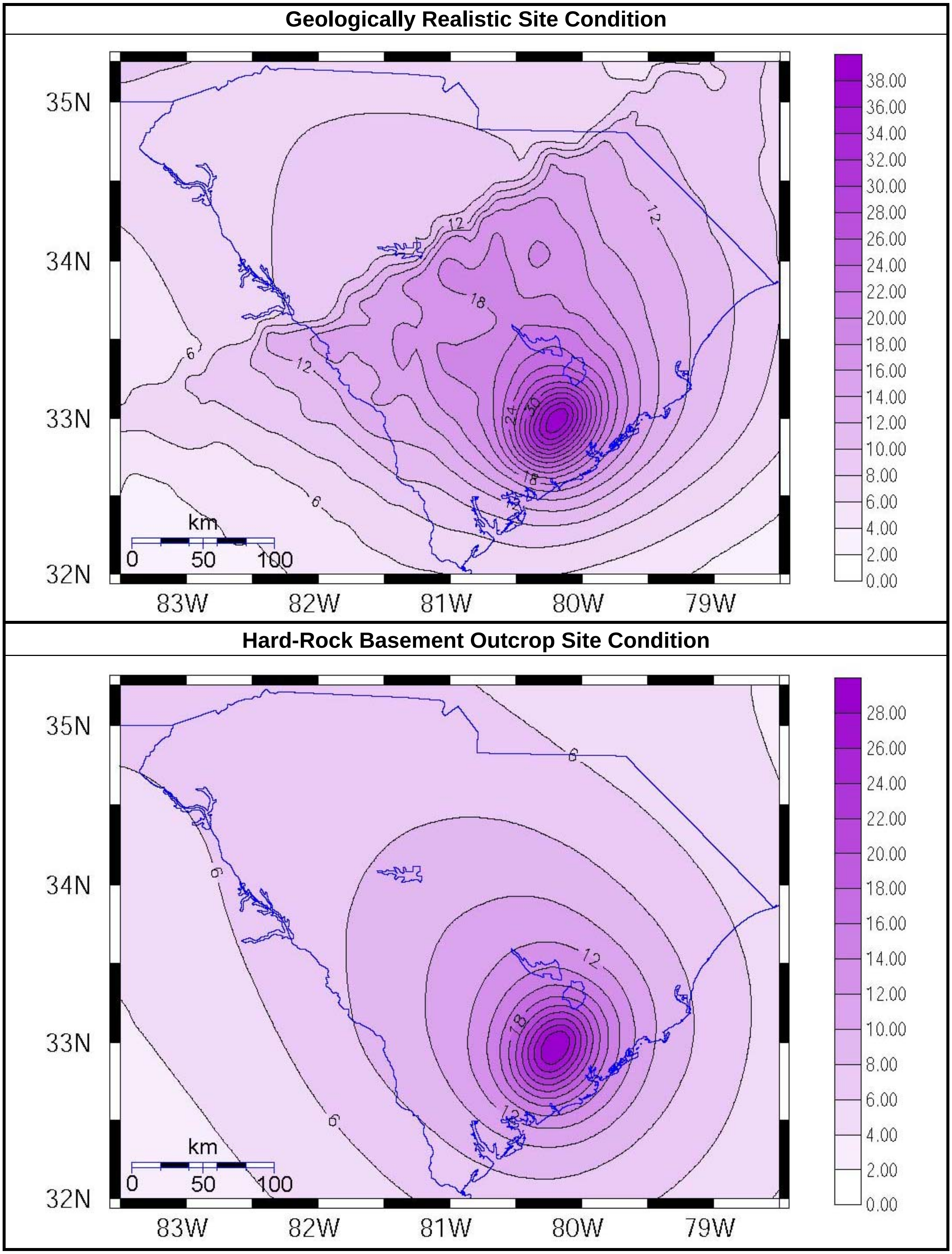
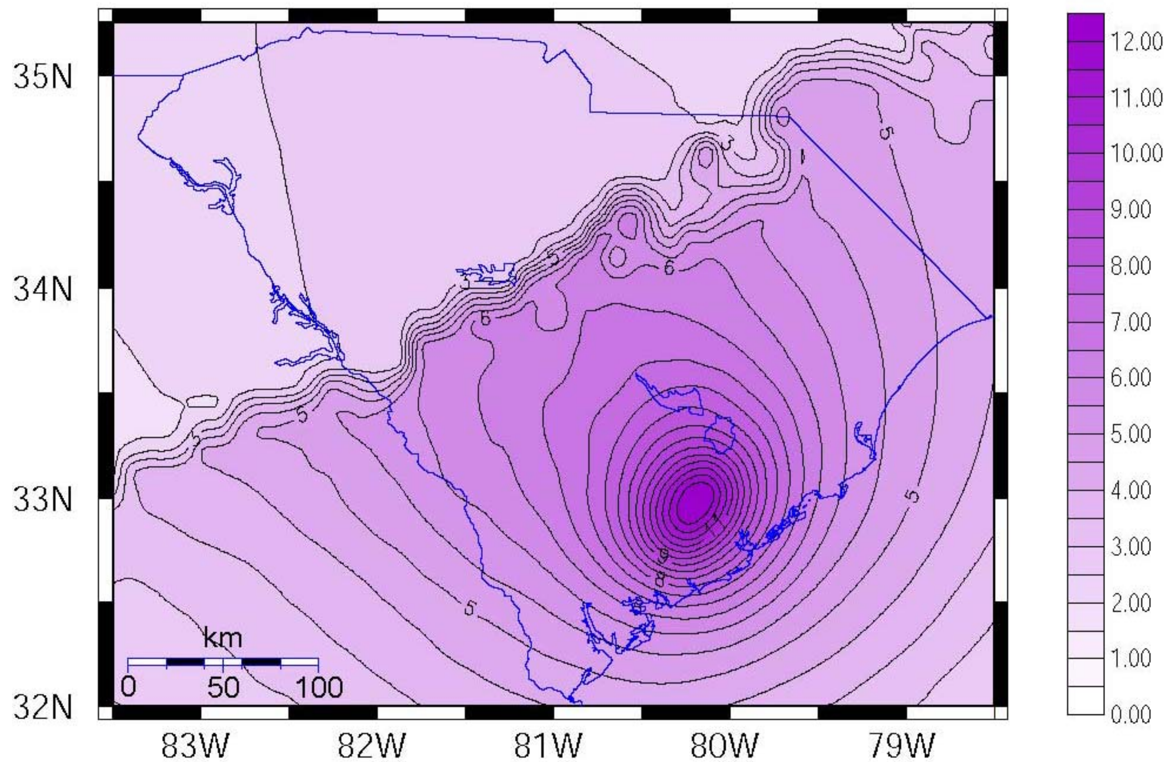


Figure 11-15, S_s Spectral Acceleration (%g) - 15% P_E in 75 Years (10% P_E-50 Yr) (Chapman and Talwani, 2002)

Geologically Realistic Site Condition



Hard-Rock Basement Outcrop Site Condition

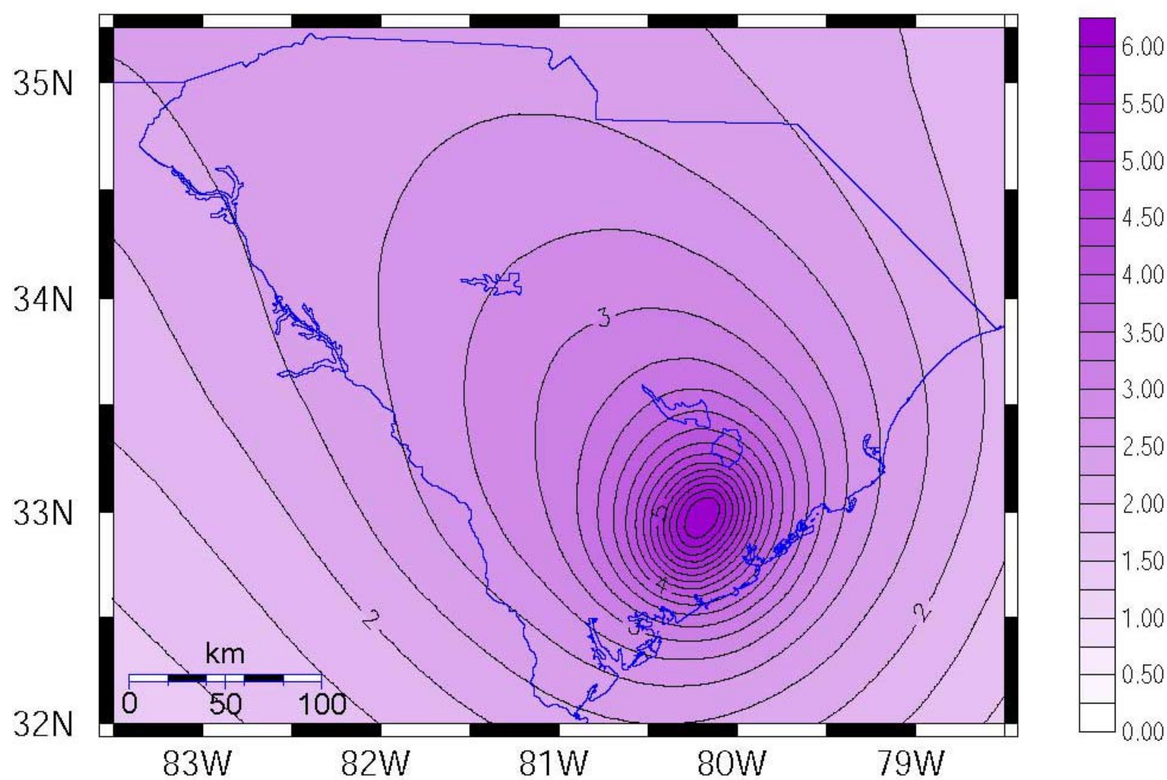


Figure 11-16, S_1 Spectral Acceleration (%g) - 15% P_E in 75 Years (10% P_E-50 Yr) (Chapman and Talwani, 2002)

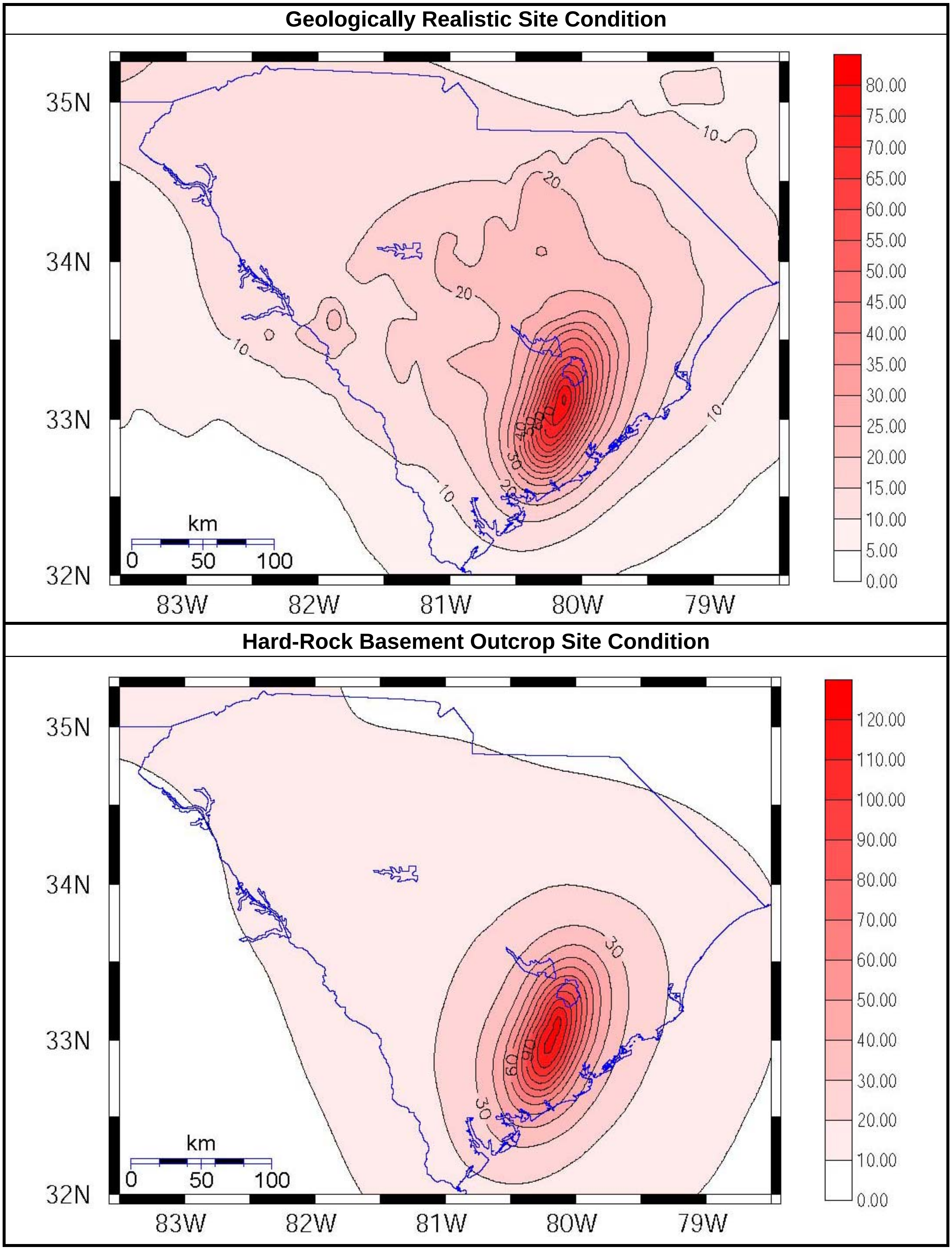
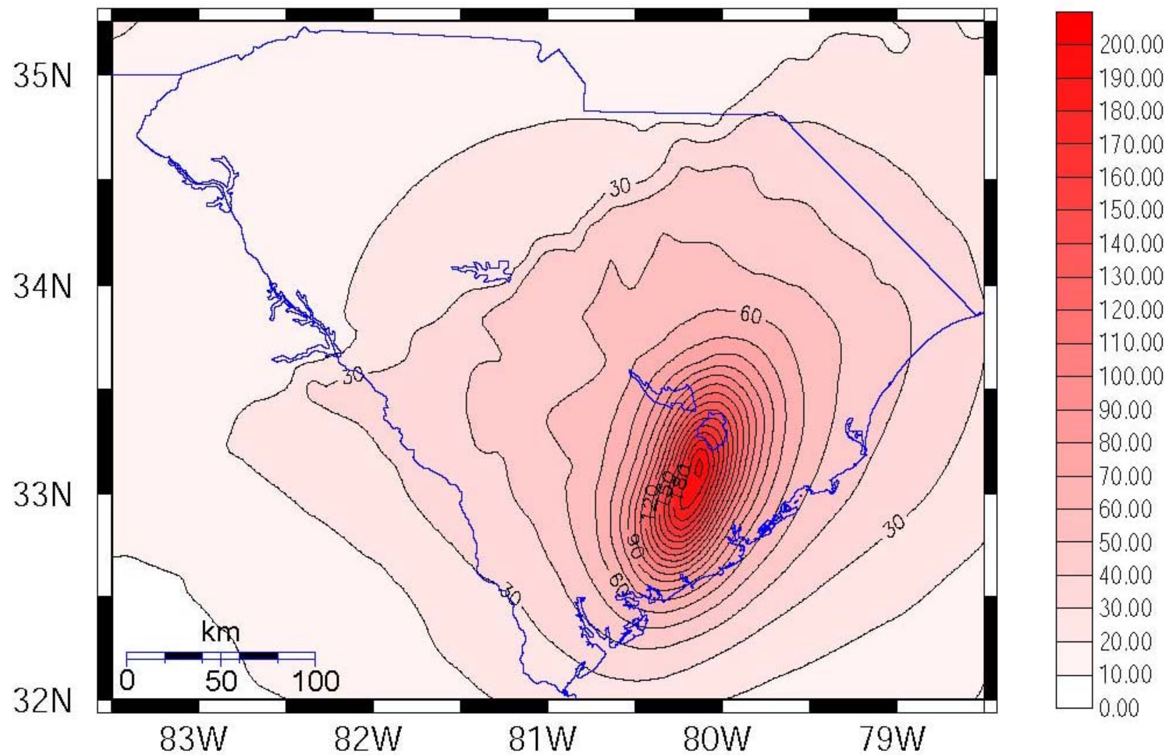


Figure 11-17, PGA (%g) - 3% P_E in 75 Years (2% P_E-50 Yr)
(Chapman and Talwani, 2002)

Geologically Realistic Site Condition



Hard-Rock Basement Outcrop Site Condition

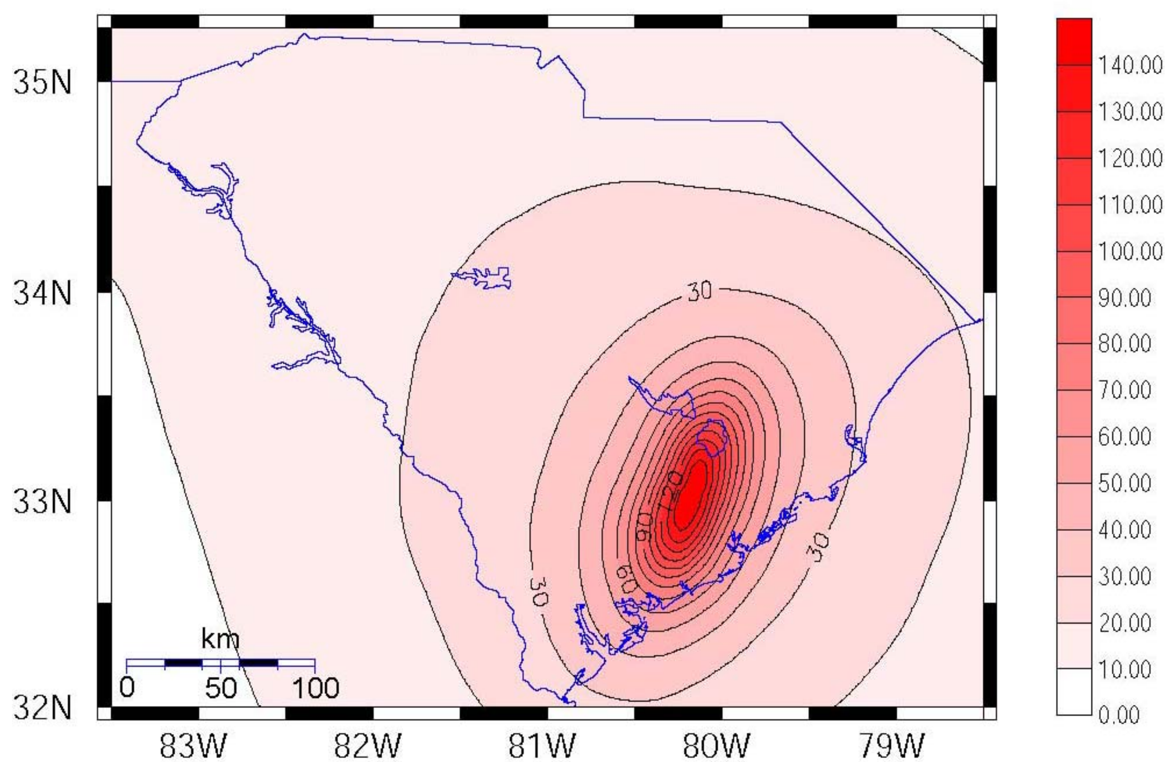
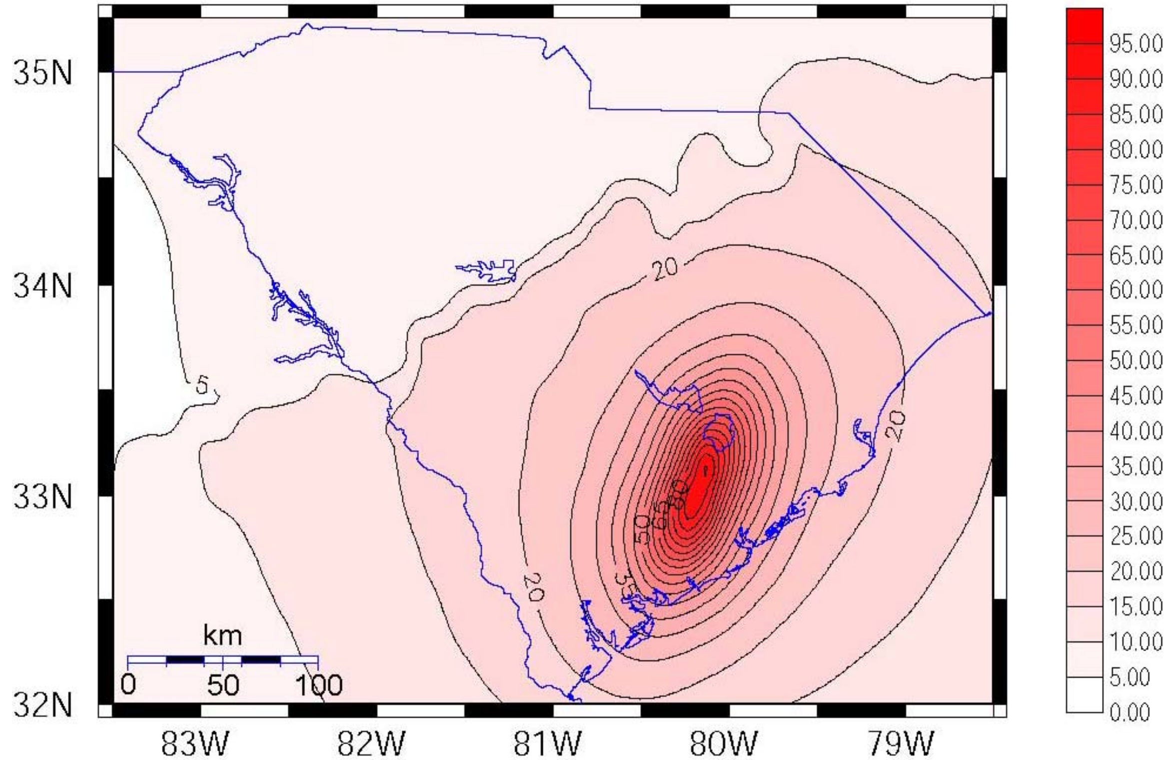


Figure 11-18, S_s Spectral Acceleration (%g) - 3% P_E in 75 Years (2% P_E-50 Yr) (Chapman and Talwani, 2002)

Geologically Realistic Site Condition



Hard-Rock Basement Outcrop Site Condition

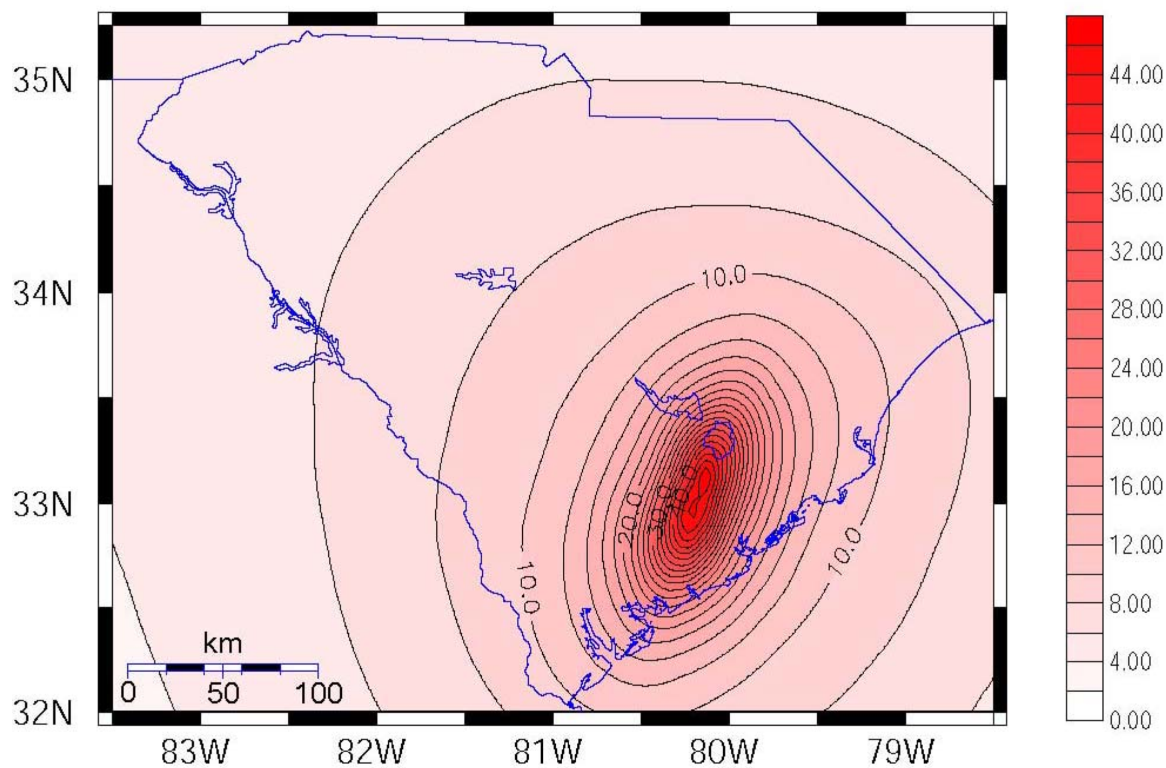


Figure 11-19, S_1 Spectral Acceleration (%g) - 3% P_E in 75 Years (2% P_E-50 Yr) (Chapman and Talwani, 2002)

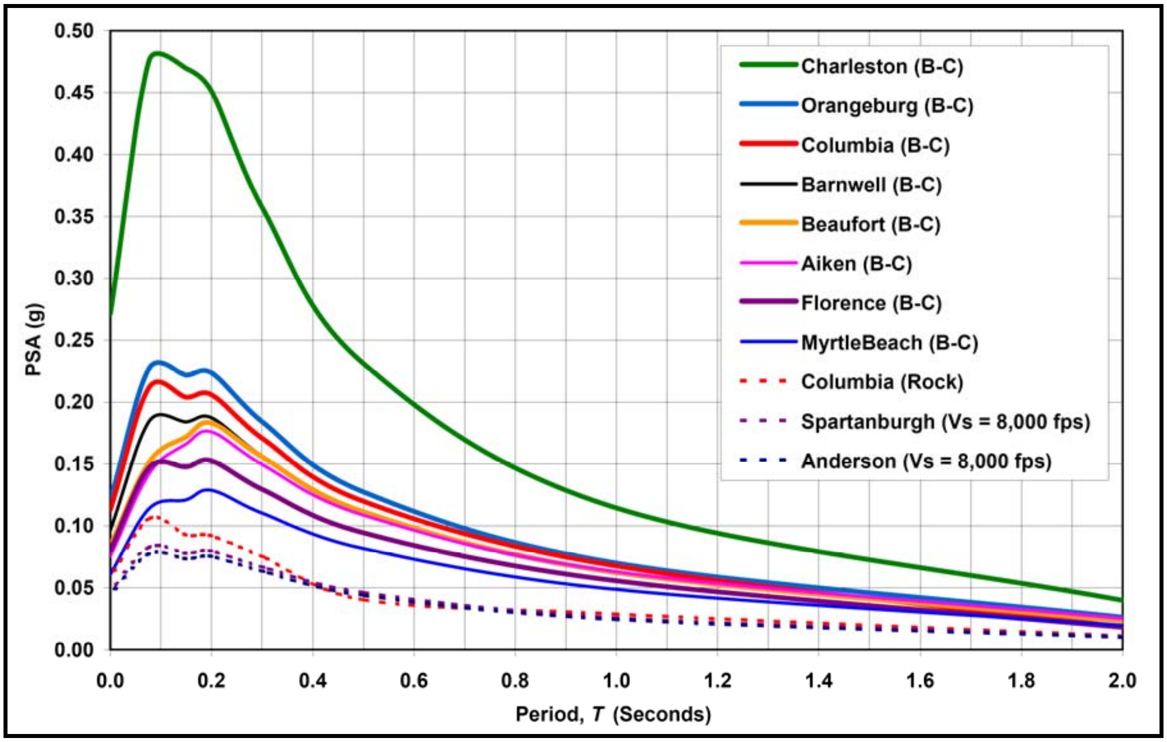


Figure 11-20, FEE PSA Curves for Selected South Carolina Cities

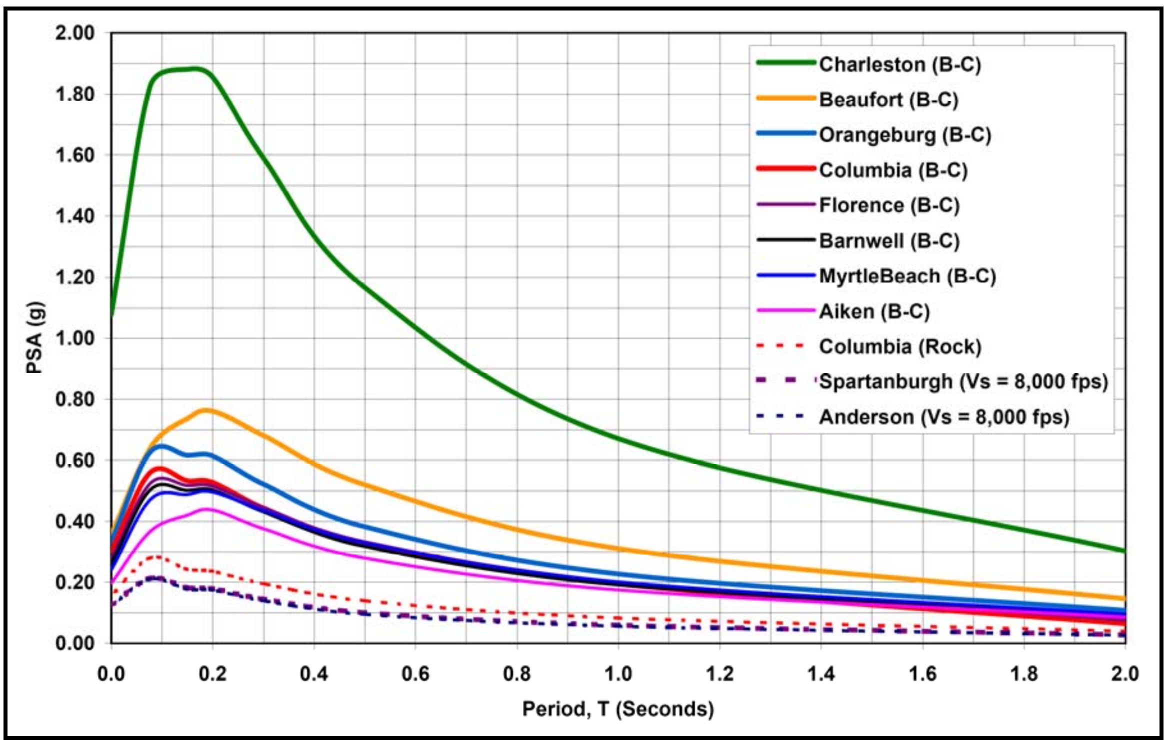


Figure 11-21, SEE PSA Curves for Selected South Carolina Cities

11.9.3 Earthquake Deaggregation Charts

The ground motion hazard from a probabilistic seismic hazard analysis can be deaggregated to determine the predominant earthquake moment magnitude (M_W) and distance (R) contributions from a hazard to guide in the selection of earthquake magnitude, site-to-source distance, and in development of appropriate time histories. The deagregation charts can be obtained by either of the following methods:

- SCDOT Scenario_PC (2006)
- USGS Interactive Earthquake Deaggregation 2002

The SCDOT Scenario_PC (2006) generates the interpolated results from the USGS Deaggregation 2002 data. A sample deaggregation output is provided in Figure 11-22 that was generated along with the SC Seismic Hazard results shown in Figure 11-13.

```
Interpolated results from USGS Deaggregation 2002
Freq. R(mean) km   mag(mean)   eps0(mean) R(modal) km  mag(modal)  eps0(modal)
PGA          58.6        6.31         .44       125.4        7.31         1.23
5 Hz         77.3        6.64         .68       125.1        7.30         1.05
1 Hz        113.1        7.06         .74       125.0        7.30          .81
```

Figure 11-22, Scenario_PC (2006) Deaggregation – Columbia, SC

Deaggregation of the seismic hazard can also be obtained from the USGS 2002 Interactive Deaggregation web site. The steps required to obtain USGS web site deaggregations are listed in Table 11-8. The project site Latitude and Longitude are obtained in the same manner as described in Section 11.9.2.

Table 11-8, USGS Interactive Deaggregation of Seismic Hazard

Step	Action
1	**Access** the USGS 2002 Interactive Deaggregations website to obtain the hazard deaggregation response for PGA and PSA frequencies. Website: http://eqint.cr.usgs.gov/deaggint/2002/index.php
2	**Complete the screen form (See Figure 11-23):** **Enter** "Site Name" **Enter** "Site Latitude and Longitude (*negative*) Coordinates" **Select** "Return period based on design earthquake": 10% PE 50 yrs = 15% PE 75 yrs (FEE) 2% PE 50 yrs = 3% PE 75 yrs (SEE) **Select** "SA Frequency": 5.0 Hz = 0.2 sec for Short-Period SA (S_S) 1.0 Hz = 1.0 sec for Long-Period SA (S_1) PGA **Select** Geographic Deaggregation – Optional (Fine Angle, Fine Distance) **Select** Stochastic Seismograms – Select None **Select** Generate Output
3	**Documents Generated:** Report - Hazard Matrix Data File (Figure 11-24) Deaggregation - Deaggregation Seismic Hazard Graph (Figure 11-25) Geographic Deaggregation – Optional (Figure 11-26)

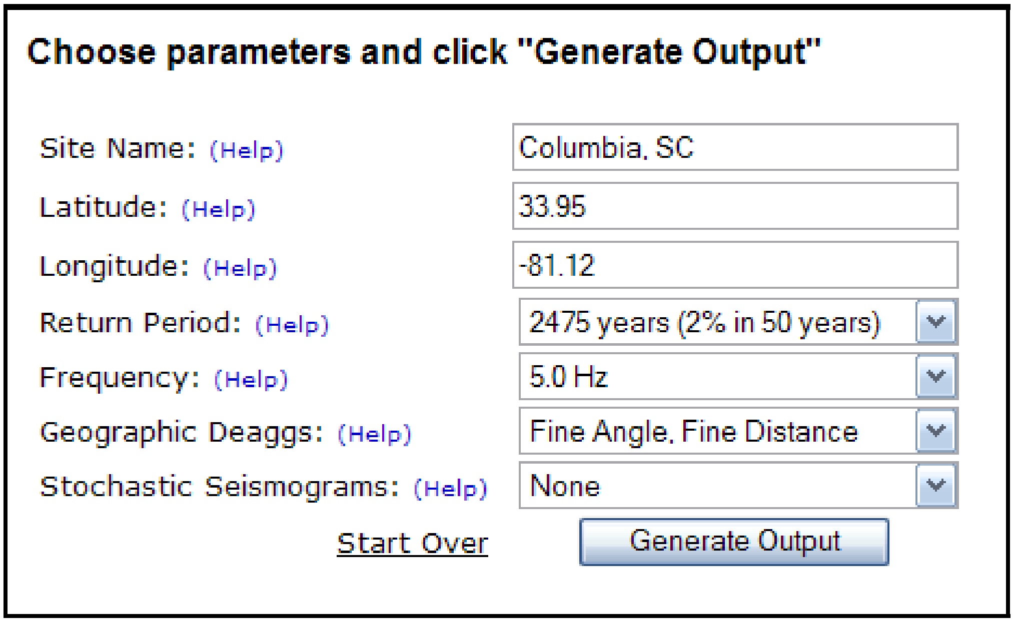


Figure 11-23, Interactive Deaggregation Input Screen (USGS 2002 Earthquake Deaggregations)

The Deaggregated Seismic Hazard Graph for the data entered in Figure 11-23 is shown in Figure 11-24. An abridged sample of the Hazard Matrix Data File is shown in Figure 11-25. The geographic deaggregation is shown in Figure 11-26.

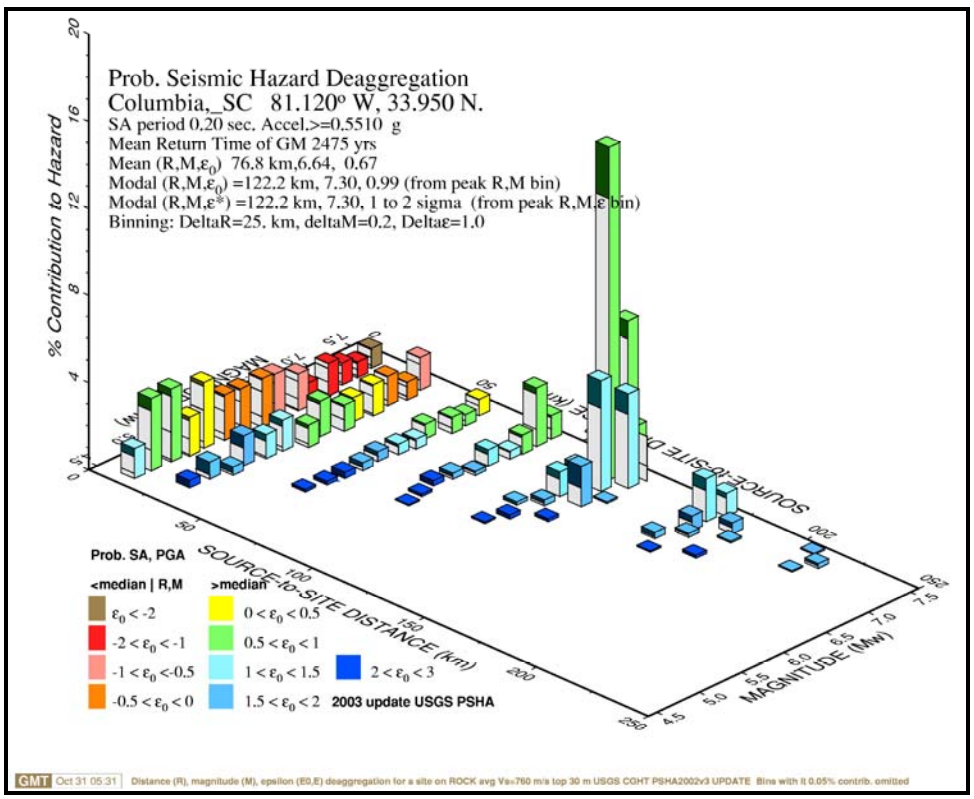


Figure 11-24, Columbia, SC Deaggregation SEE (3% P_E in 75 Years, 1Hz PSA) (USGS 2002 Earthquake Deaggregations)

```
********************Central or Eastern U.S. Site ********************************
PSHA Deaggregation. %contributions. ROCK site: Columbia, _SC  long:  81.120 d W.,
                              lat: 33.950 N.
     USGS 2002-2003 update files and programs. Analysis on DaMoYr:31/10/2006
                Return period: 2475 yrs. 0.20 s. PSA =0.5510   g.
          #Pr[at least one eq with median motion>=PSA in 50 yrs]=0.00397
  DIST(km) MAG(Mw) ALL_EPS EPSILON>2  1<EPS<2 0<EPS<1 -1<EPS<0 -2<EPS<-1 EPS<-2
   181.3    7.18    0.080    0.042    0.039    0.000    0.000    0.000    0.000
   209.8    7.15    0.056    0.043    0.013    0.000    0.000    0.000    0.000
    12.8    7.39    0.915    0.001    0.061    0.354    0.361    0.131    0.007
    34.7    7.39    1.495    0.033    0.283    0.712    0.443    0.023    0.000
    60.8    7.39    0.753    0.042    0.268    0.434    0.009    0.000    0.000
    89.2    7.32    2.696    0.287    1.621    0.788    0.000    0.000    0.000
   122.2    7.30   15.286    2.376   11.030    1.879    0.000    0.000    0.000
   130.9    7.30    4.347    0.958    3.389    0.000    0.000    0.000    0.000

                 . . .  Additional output data omitted  . . .

   Summary statistics for above 0.2s PSA  deaggregation, R=distance, e=epsilon:
    Mean src-site R=   76.8 km; M= 6.64; eps0=   0.67. Mean calculated for all
                                 sources.
      Modal src-site R=  122.2 km; M= 7.30; eps0=   0.99 from peak (R,M) bin
               Gridded source distance metrics: Rseis Rrup and Rjb
    MODE R*= 122.2km; M*= 7.30; EPS.INTERVAL: 1 to 2 sigma  % CONTRIB.= 11.030

Principal sources (faults, subduction, random seismicity having >10% contribution)
Source Category:                          % contr.   R(km)    M   epsilon0 (mean
                                 values)
          Charleston Broad Zone             19.64   126.9   7.29    1.09
          Charleston Narrow Zone            24.38   125.0   7.29    1.06
          CEUS gridded seism.               55.98    38.2   6.12    0.36
                 Individual fault hazard details if contrib.>1%:
********************Central or Eastern U.S. Site ********************************
```

Figure 11-25, Abridged Seismic Hazard Matrix Data – Columbia, SC (USGS 2002 Earthquake Deaggregations)

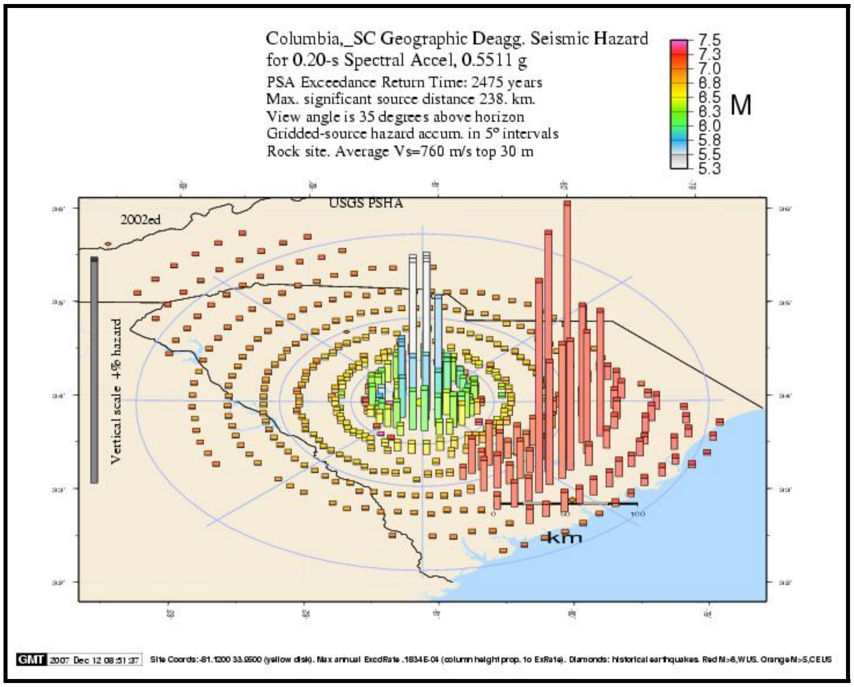


Figure 11-26, Geographic Deaggregation (Optional) (USGS 2002 Earthquake Deaggregations)

The earthquake deaggregations typically provide the source category, percent contribution of the source to the hazard, site-to-source distance (R), mean and modal moment magnitude (M), and epsilon (ε). Mean moment magnitudes (M_W) that cover several sources are typically not used since it is an overall average of earthquakes and does not appropriately reflect magnitude of the hazard contribution within a specific seismic source. Mean moment magnitude (M_W) values listed with respect to principal sources can be used. The epsilon (ε) parameter is as important to understanding a ground motion as is the moment magnitude (M_W) and the distance (R) values for the various sources. The epsilon (ε) parameter is a measure of how close the ground motion is to the mean value in terms of standard deviation (σ). The epsilon ε_o parameter is provided for ground motions having a fixed probability of exceedance (P_E). If a structure is designed for an earthquake with magnitude M_W that occurs a distance R from your site and the $\varepsilon_o = 0.0$, then the structure was designed to resist a median motion from this source. If the $\varepsilon_o = 1.0$, then the structure was designed to resist a motion one standard deviation ($+1\sigma$) greater than the median motion. Consequently, if the $\varepsilon_o = -1.0$, then the structure was designed to resist a motion one standard deviation (-1σ) less than the median motion. Predominance of a modal earthquake source is generally indicated if the epsilon (ε) is within ± 1 standard deviation ($\pm 1\sigma$).

For additional information on the interpretation of the deaggregation data, the designer should refer to the information provided at the USGS 2002 Interactive Deaggregation web site. The method chosen to deaggregate the South Carolina seismic hazard should be based on the intended use of the deagregation data. For example, the Scenario_PC (2006) deaggregations are sufficient to select the earthquake moment magnitude (M_W) and site-to-source distance (R) for liquefaction potential analyses and lateral spreading analyses. When performing a site-specific response analysis, the 2002 USGS Interactive Deaggregations are more detailed and informative and should therefore be used to obtain the earthquake moment magnitude (M_W) and site-to-source distance (R) used to generate the ground motion time histories. Further guidance in the method of obtaining and interpreting the earthquake deagregation data is provided in Chapter 12 and in Section 11.9.4, Ground Motions.

11.9.4 <u>Ground Motions</u>

Ground motions are required when a site-specific design response analysis and/or a site-specific seismic deformation analysis is being performed. These ground motions are developed from a site-specific ground shaking characterization that generates a time history. Time histories can be either recorded with seismographs or synthetically developed. Since the Charleston 1886 earthquake occurred, an earthquake with a magnitude of +7 has not occurred in South Carolina and therefore no seismograph records are available for strong motion earthquakes in South Carolina. SCDOT has chosen to generate synthetic project-specific time histories based on the SC Seismic Hazard study recently completed for SCDOT. The ground motion predictions used in the study are based on the results of recent work involving both empirical and theoretical modeling of Eastern North American strong ground motion. Even though the strong motion database for the East is small compared to the West, the available data indicate that high frequency ground motions attenuate more slowly in the East than in the West.

Synthetic ground motions can be developed using an attenuation model. The ground motions on hard rock produced from the SCDOT Seismic Hazard program Scenario_PC (2006) uses a stochastic model that uses weighted (w) attenuation relationships from 1987 Toro et al. (w=0.143), 1996 Frankel et al. (w=0.143), 1995 Atkinson and Boore (w=0.143), 2001 Somerville et al. (w=0.286), and 2002 Campbell (w=0.286) for the characteristic earthquake events with magnitudes ranging from 7.0 to 7.5. For the non-characteristic earthquake events with magnitudes less than 7.0, the following weighted prediction equations were used, 1977 Toro et al. (w=0.286), 1996 Frankel et al. (w=0.286), 1995 Atkinson and Boore (w=0.286), and 2002 Campbell (w=0.143).

The location of the ground motion is dependent on the Site Condition (Geologically Realistic or Hard-Rock Basement Outcrop) selected in Section 11.9.2. Table 11-9 provides the location where the ground motions are computed based on the Site Condition selected and Geologic Unit.

Table 11-9, Location of Ground Motion

Site Condition	Geologic Unit [1]	Location of Ground Motion
Geologically Realistic	Piedmont / Blue Ridge (Zone I)	Generated at a hypothetical outcrop of weathered rock (Vs = 8,200 ft/sec) equivalent to Site Class A (Vs > 5,000 ft/sec)
	Coastal Plain (Zone II)	Generated at a hypothetical outcrop of firm Coastal Plain sediment (V_s = 2,500 ft/sec) equivalent to the B – C Boundary
Hard-Rock Basement Outcrop	Piedmont / Blue Ridge (Zone I)	Generated at a hard-rock basement outcrop (V_s = 11,500 ft/sec) equivalent to Site Class A (Vs > 5,000 ft/sec)
	Coastal Plain (Zone II)	

[1] For geologic unit locations see Figure 11-1 and 11-3 and for Site Condition locations see Figure 11-12.

The time histories are generated based on project specific information using Scenario_PC (2006). The consultant must submit a SC Ground Motion request form to the GDS to obtain project specific time histories. The SC Ground Motion request form requires that the designer provide the following information.

- SCDOT Project Name and Project Number
- Latitude and Longitude of Project Site
- Probability of Exceedance for Earthquake Design Event being analyzed
- Site Condition: Geologically Realistic or Hard-Rock Basement Outcrop
- Sediment Thickness: If other than default thickness generated from Scenario_PC
- Scaling Method: Scaling of the time series to match Uniform Hazard, PGA, or PSA
- Moment magnitude (M_w) and epicenter site-to-source distance (R)

The sediment thickness may be changed from the default value if a site-specific geotechnical investigation indicates that the sediment thickness is different from the value generated in the Scenario_PC (2006) output.

The method of scaling the time series to match a Uniform Hazard Spectrum (UHS), PGA, or a PSA frequency is primarily dependent on the results of the earthquake deaggregation described in Section 11.9.3. When the uniform hazard is dominated by a well-defined modal earthquake event, the method of scaling the time series should be to match the UHS.

The Coastal Plain will typically be dominated by the 1886 Charleston earthquake seismic source as can be seen in Figure 11-27, Florence, SC Deaggregation FEE (USGS 2002). The earthquake deagregation chart in Figure 11-27 indicates that the FEE 1Hz PSA design earthquake would have a modal source site with a Moment Magnitude (M_w) of 7.30 with an epicenter site-to-source distance (R) of 87.1 km and an epsilon (ε_o) parameter of –0.85. The SEE 1Hz PSA design earthquake for Florence, SC in Figure 11-28 indicates a modal source site with a Moment Magnitude (M_w) of 7.30 with an epicenter site-to-source distance (R) of 36.2 km and an epsilon (ε_o) parameter of 0.01. As a result of the predominance of the 1886 Charleston Earthquake seismic source in the Coastal Plain geological unit, the time series generated for most project sites in the Coastal Plain should be scaled to match the UHS. By contrast, the FEE and SEE Anderson, SC Deaggregation (USGS 2002), shown in Figures 11-29 and 11-30, respectively, show several earthquakes that may be of significance to evaluating seismic hazards at the project site. Table 11-10 provides a summary of FEE 1Hz potential seismic sources that may be used for scaling the time series. All FEE 1Hz seismic sources appear to be equally predominant epsilons (ε) within ± 1 standard deviation ($\pm 1\sigma$).

Table 11-10, FEE 1Hz PSA Deaggregation Summary - Anderson, SC

Seismic Source Site	% Contribution	R Distance, km	M_W	ε_O
1886 Charleston Seismic Source	28.7	235	7.26	0.19
New Madrid (NMSZ)	12.5	640	7.72	0.82
CEUS	58.8	184	6.52	0.34
Total Contribution % =	100.0	---	---	---
Modal Source Site	---	282	7.30	0.09

Table 11-11 provides a summary of SEE 1Hz potential seismic sources that may be used for scaling the time series. The SEE 1Hz CEUS seismic source site appears to be predominate with an epsilons (ε) of 0.65.

Table 11-11, SEE 1Hz PSA Deaggregation Summary - Anderson, SC

Seismic Source Site	% Contribution	R Distance, km	M_W	ε_O
1886 Charleston Seismic Source	23.70	238	7.30	1.21
New Madrid (NMSZ)	6.9	644	7.77	1.77
CEUS	64.4	125	6.72	0.65
Total Contribution % =	100.0	---	---	---
Modal Source Site	---	282	7.30	1.17

Similar deaggregation data can be obtained for PGA or other PSA frequencies. Based on the type of structure being designed or seismic hazard being analyzed, there may be a need to develop more than one earthquake seismic source time series and have it matched to the PGA or a PSA frequency.

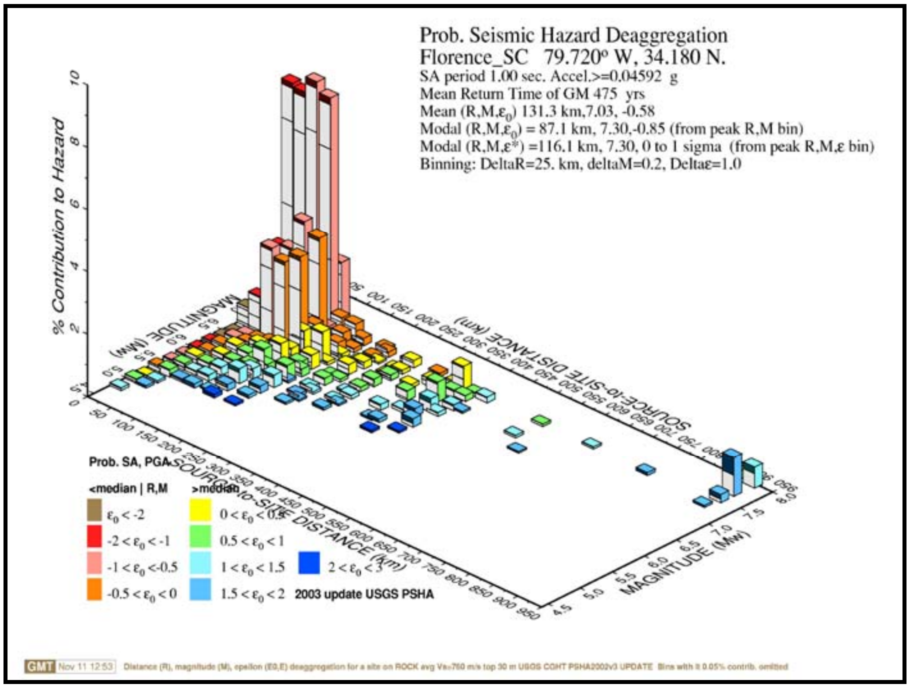


Figure 11-27, Florence, SC Deaggregation FEE (15% P_E in 75 Years, 1Hz PSA) (USGS 2002 Earthquake Deaggregations)

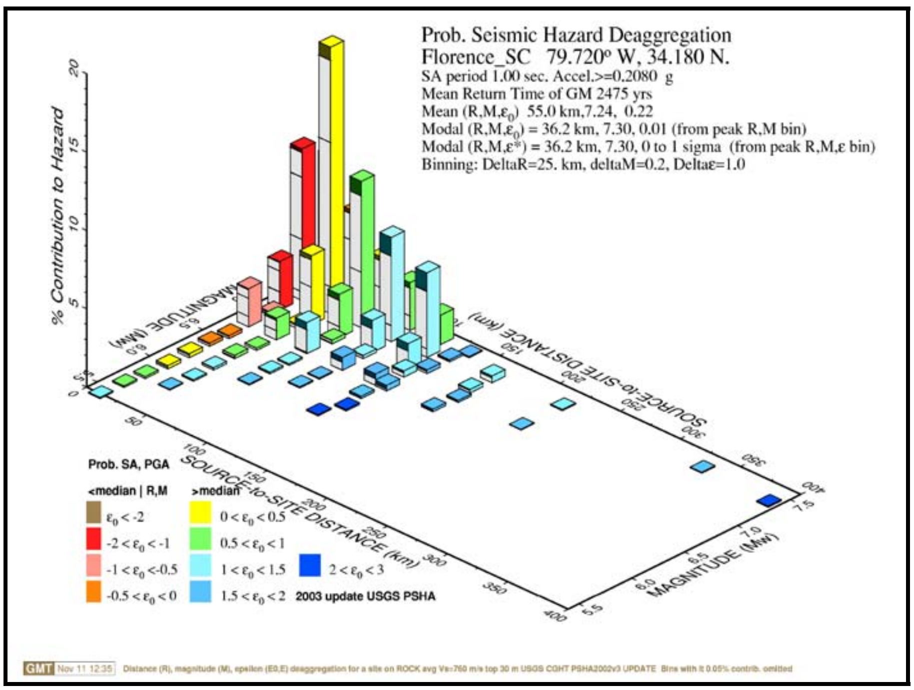


Figure 11-28, Florence, SC Deaggregation SEE (3% P_E in 75 Years, 1Hz PSA) (USGS 2002 Earthquake Deaggregations)

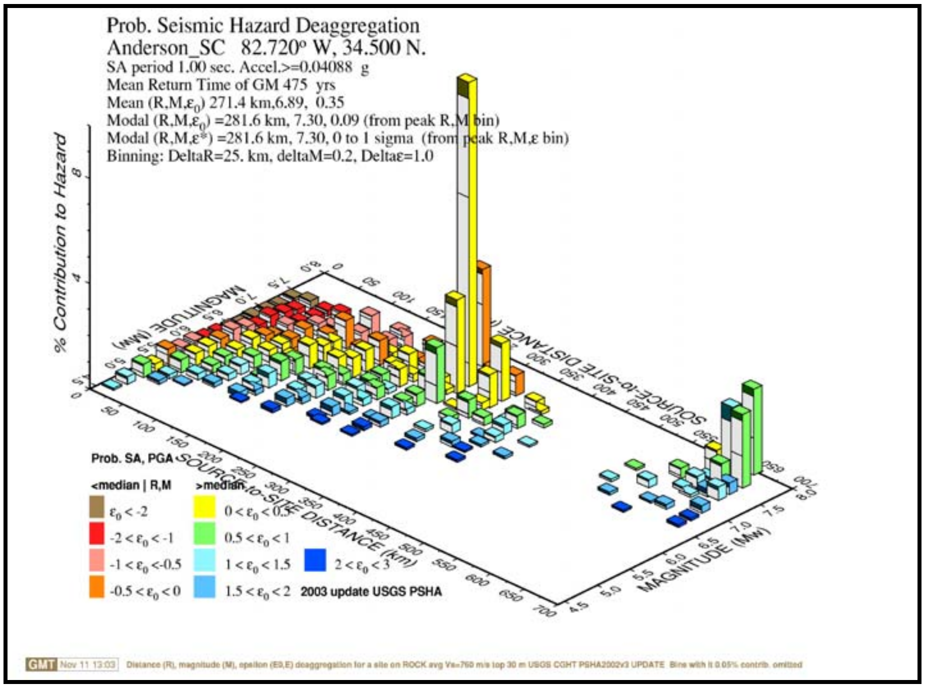


Figure 11-29, Anderson, SC Deaggregation FEE (15% P_E in 75 Years, 1Hz PSA) (USGS 2002 Earthquake Deaggregations)

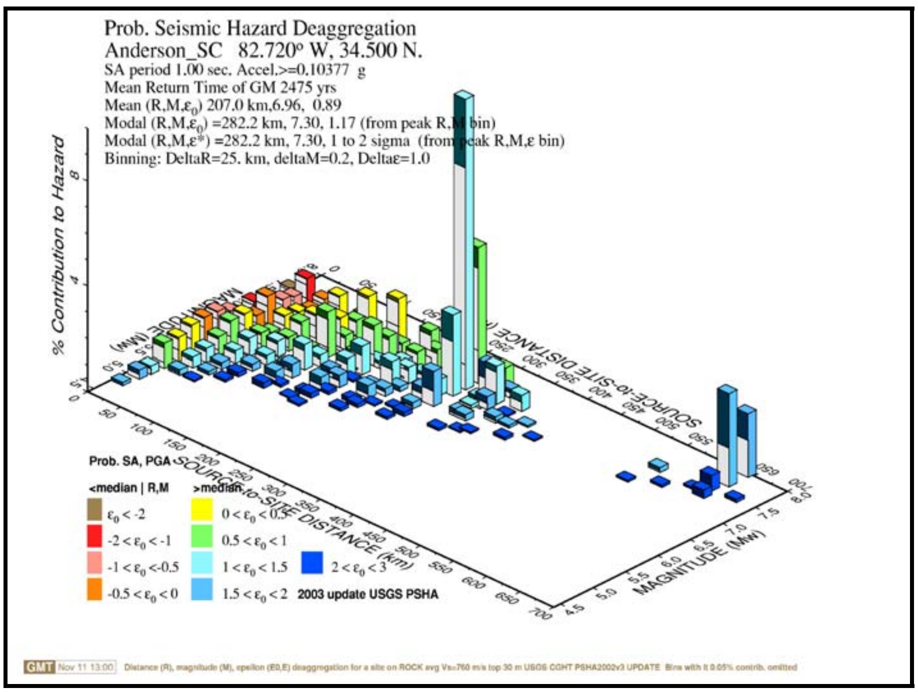


Figure 11-30, Anderson, SC Deaggregation SEE (3% P_E in 75 Years, 1Hz PSA) (USGS 2002 Earthquake Deaggregations)

11.10 REFERENCES

The geotechnical information contained in this Manual must be used in conjunction with the SCDOT *Seismic Design Specifications for Highway Bridges*, SCDOT *Bridge Design Manual*, and AASHTO LRFD Bridge Design Specifications. The Geotechnical Design Manual will take precedence over all references with respect to geotechnical engineering design.

AASHTO LRFD Bridge Design Specifications, U.S. Customary Units, 4th Edition, (2007), American Association of State Highway and Transportation Officials.

Atkinson, G.A. and D.M. Boore, (1995), "Ground motion relations for Eastern North America", Bulletin of the Seismological Society of America, v. 85, pp. 17-30.

Boore D.M. and W.B. Joyner, (1991), "Ground motion at deep soil sites in eastern North America", Bulletin of the Seismological Society of America, v. 81, pp. 2167-2187.

Campbell, K. W., (2002). "Prediction of strong ground motion using the hybrid empirical method: example application to eastern North America", submitted to Bull. Seism.
Soc. Am.

Chapman and Talwani (2002). "Seismic Hazard Mapping for Bridge and Highway Design", SCDOT, Referred to in this manual as SCDOT Seismic Hazard Study.

Chapman (2006). "User's Guide to SCENARIO_PC and SCDOTSHAKE", SCDOT.

Frankel, A., C. Mueller, T. Barnhard, D. Perkins, E.V. Leyendecker, N. Dickman, S. Hanson and M. Hopper, (1996), "National seismic-hazard maps; documentation", United States Geological Survey Open-File Report 96-532, 110p.

Marple, R.T., and P. Talwani, (1993), "Evidence of possible tectonic upwarping along the South Carolina Coastal Plain from an examination of river morphology and elevation data", Geology, v. 21, pp. 651-654.

Marple, R.T., and P. Talwani, (2000), "Evidence for a buried fault system in the Coastal Plain of the Carolinas and Virginia; Implications for neotectonics in the southeastern United States", Geological Society of America Bulletin, v. 112, no. 2, pp. 200-220.

Odum, J.K., Williams, R.A., Stepheson, W.J., and Worley, D.M., (2003). "Near-surface S-wave and P-wave seismic velocities of primary geological formations on the Piedmont and Atlantic Coastal Plain of South Carolina, USA", United States Geological Survey Open-File Report 03-043, 14p.

Snipes, D.S.,W.C. Fallaw, Van Price Jr. and R.J. Cumbest, (1993). "The Pen Branch Fault: Documentation of Late Cretaceous-Tertiary Faulting in the Coastal Plain of South Carolina", *Southeastern Geology*, 33, No.4, 195-218

South Carolina Department of Natural Resources, Geologic Survey, (2005). "Generalized Geologic Map of South Carolina 2005," http://www.dnr.state.sc.us/geology/geology.htm

South Carolina Department of Natural Resources, Geologic Survey. “Geologic Time Scale for South Carolina”, http://www.dnr.state.sc.us/geology/images/Time%20scale_pagesize.pdf

South Carolina Department of Natural Resources, Geologic Survey. “Earthquake Intensities By County”, http://www.dnr.state.sc.us/geology/earthquake_intens.htm

SCDOT *Bridge Design Manual* (2006), South Carolina Department of Transportation, http://www.scdot.org/doing/bridge/06design_manual.shtml

SCDOT *Seismic Design Specifications for Highway Bridges* (2008), South Carolina Department of Transportation, http://www.scdot.org/doing/bridge/bridgeseismic.shtml

Somerville, P., N. Collins, N. Abrahamson, R. Graves and C. Saikia, (2001), “Ground motion attenuation relations for the central and eastern United States”, final report to the U. S. Geological Survey.

Toro, G., and R.K. Mcguire, (1987), “An investigation into earthquake ground motion characteristics in eastern North America”, Bulletin of the Seismological Society of America, v. 77, pp. 468-489.

United States Geological Service (USGS), “Damaging Earthquakes In the US (1750 – 1996)”, http://earthquake.usgs.gov/regional/states/us_damage_eq.php

Chapter 12

GEOTECHNICAL EARTHQUAKE ENGINEERING

FINAL

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Table of Contents

List of Tables

List of Figures

CHAPTER 12

GEOTECHNICAL EARTHQUAKE ENGINEERING

12.1 INTRODUCTION

Geotechnical earthquake engineering consists of evaluating the earthquake hazard and the effects of the hazard on the transportation structure being designed. This is accomplished by characterizing the subsurface soils, determining the earthquake hazard, evaluating the local site effects on the response spectra, and developing an acceleration design response spectrum (ADRS) for use in designing bridges and other transportation structures.

SCDOT has made a commitment to design transportation systems in South Carolina so as to minimize their susceptibility to damage from earthquakes. The SCDOT *Seismic Design Specifications for Highway Bridges* establishes the seismic design requirements for the design of bridges in the South Carolina highway transportation system. This chapter presents geotechnical earthquake engineering design requirements for evaluating ground shaking using either SC Seismic Hazard maps or by performing a site-specific response analysis. The SC Seismic Hazard Maps and Deaggregation Charts are discussed in Chapter 11. Geotechnical seismic analysis and design guidelines for evaluating soil liquefaction potential, analyzing liquefaction induced hazards, seismic slope stability, and analyzing seismic lateral loadings are contained in Chapters 13 and 14.

The GDS performs the following types of geotechnical earthquake engineering analyses:

1. Geotechnical Seismic Site Characterization (Chapter 12)
2. Performs Earthquake Hazard Analyses – Liquefaction, etc. (Chapter 13)
3. Generates Earthquake Ground Motions - Time Histories (Chapter 11)
4. Determines Earthquake Design Parameters – PGA, PSA, M_w, etc. (Chapter 11)
5. Develops Acceleration Design Response Spectrum (ADRS) curves (Chapter 12)
6. Develops Geotechnical Earthquake Engineering Design Guidelines (Chapter 14)
7. Reviews Consultant Geotechnical Earthquake Engineering Reports (Chapter 3)

12.2 GEOTECHNICAL EARTHQUAKE ENGINEERING DESIGN

The geotechnical earthquake engineering requirements for determining the seismic hazard and associated response have been developed for the design of "Typical SCDOT Bridges" as defined by Sections 1.4 and 1.5 of the SCDOT *Seismic Design Specifications for Highway Bridges*. Bridges not meeting the definition of "Typical SCDOT Bridges" include suspension bridges, cable-stayed bridges, arch type bridges, movable bridges, and bridge spans exceeding 300 feet. For these non-typical bridges, the PCS/GDS will specify and/or approve appropriate geotechnical earthquake engineering provisions on a project specific basis. The geotechnical earthquake engineering requirements in this Manual also apply to the design of geotechnical roadway structures such as roadway embankments, earth-retaining systems, and other miscellaneous transportation related structures.

The preliminary geotechnical engineering report (PGER) typically contains a geotechnical earthquake hazard analysis that includes the determination of a Site Class based on available subsurface information and a horizontal acceleration design response spectrum (ADRS) to be used for preliminary design of the bridge structure. The final geotechnical engineering report (BGER or RGER) contains the results of the final geotechnical subsurface investigation and modifies, if necessary, the Site Class and the horizontal acceleration design response spectrum (ADRS) curves.

12.3 DYNAMIC SOIL PROPERTIES

12.3.1 <u>Soil Properties</u>

A project specific subsurface geotechnical investigation is typically required in accordance with the subsurface investigation guidelines provided in Chapter 4. Basic soil properties will be obtained in accordance with the field and laboratory testing procedures specified in Chapter 5. Basic soil properties can be directly measured by field and laboratory testing results or can be correlated from those results as described in Chapter 7. Dynamic soil properties such as shear wave velocity, V_S, should be measured in the field (Chapter 5) and correlated as indicated in this Chapter when insufficient field measurements are available. Other dynamic properties such as shear modulus curves, equivalent viscous damping ratio curves, and residual strength of liquefied soils are determined as indicated in this Chapter.

12.3.2 <u>Soil Stiffness</u>

One of the required soil properties needed to perform a soil response analysis is the soil stiffness. Soil stiffness is characterized by either small-strain shear-wave velocity or small-strain shear modulus. The small-strain shear wave velocity, V_S, is related to small-strain shear modulus, G_{max}, by the following equation.

$$\boldsymbol{G}_{\max} = \rho \boldsymbol{V_S}^2 = \frac{\gamma_T \boldsymbol{V_S}^2}{\boldsymbol{g}} \qquad \text{Equation 12-1}$$

Where the mass density of soil, ρ, is equal to the total unit weight, γ_T, of the soil divided by the acceleration of gravity ($g = 32.174$ ft/sec^2 = 9.81 m/sec^2).

Typical values of small-strain shear wave velocity, V_S, and small-strain shear modulus, G_{max}, for various soil types are shown in Table 12-1. Additional guidance on selecting appropriate shear wave velocities can be obtained by reviewing the database range of shear wave velocities for different South Carolina soil deposits indicated in Tables 12-3 and 12-9. Typical small-strain shear wave velocity profiles for different parts of South Carolina are provided in Section 12.3.3.

Table 12-1, Typical Small-Strain Shear Wave Velocity and Initial Shear Modulus
(Based on Hunt, 1984 and Kavazanjian, 1998)

Soil Type	Mass Density, ρ	Total Unit Weight, γ	Small-strain Shear Wave Velocity, V_S		Initial Shear Modulus, G_{max}	
	kg/m^3	pcf	m/s	ft/s	kPa	psi
Soft Clay	1,600	100	40 – 90	130 – 300	2,600 – 13,000	400 – 2,000
Stiff Clay	1,680	105	65 – 140	210 – 500	7,000 – 33,000	1000 – 5,700
Loose Sand	1,680	105	130 – 280	420 – 920	28,400 – 131,700	4,000 – 19,200
Dense Sand and Gravel	1,760	110	200 - 410	650 – 1,350	70,400 – 300,000	10,000 – 43,300
Residual Soil (PWR, IGM)	2,000	125	300 - 600	1,000 – 2,000	180,000 – 720,000	27,000 – 108,000
Piedmont Metamorphic and Igneous Rock (Highly – Moderately Weathered)	2,500	155	760 – 3,000	2,500 – 10,000	1,400,00 – 22,500,000	209,000 – 3,400,000
0 <RQD < 50			600	2,000		
RQD = 65 [(1)]			760	2,500		
RQD = 80 [(1)]			1,500	5,000		
RQD = 90 [(1)]			2,500	8,000		
RQD = 100 [(1)]			3,400	11,000		
Basement Rock (Moderately Weathered to Intact)	2,600	165	> 3,400	> 11,000	> 30,000	> 4,300,000

[(1)] Typical Values, Linear interpolate between RQD values

When performing a geotechnical subsurface investigation it is typically preferred to measure site-specific small-strain shear wave velocity, V_S, as described in Chapters 4 and 5. When site-specific shear wave velocities, V_S, are not available or needs to be supplemented, an estimation of the shear wave velocity, V_S, can be made by the use of correlations with in-situ testing such as the Standard Penetration Test (SPT) or the Cone Penetration Test (CPT). Procedures for estimating dynamic properties of soils in South Carolina have been developed by Andrus et al. (2003). The procedures for correlating SPT and CPT results with shear wave velocity, V_S, have been summarized in Sections 12.3.2.1 and 12.3.2.2, respectively. For a more detailed description of the procedures to estimate dynamic properties see Andrus et al. (2003). A review of SPT calculated shear wave velocity relationships reveals that few relationships have been developed for clays. This is likely due to SPT blow counts (N) not being the appropriate test for cohesive soils, particularly since soft clays would have SPT blow counts that would be close to zero.

The SPT correlations for shear wave velocity, V_S, use the standardized SPT blow count, N^*_{60}, that is defined in Chapter 7. The CPT correlations for shear wave velocity, V_S, use the measured CPT tip resistance, q_c, that is defined in Chapter 5.

12.3.2.1 SPT - Shear Wave Velocity, V_S, Estimation of SC Sands

Recommended equations to estimate shear wave velocities, V_S, for South Carolina soils are based on standardized SPT blow count (N^*_{60}), depth (Z), Fines Content (FC), geologic age and location of deposit, and Age Scaling Factor (ASF). Equations for estimating shear wave velocities, V_S, of South Carolina sands are provided in Table 12-2 and shown in Figure 12-1.

Table 12-2, SPT (N^*_{60}) - Shear Wave Velocity, V_S, Equations for SC Sand (Andrus et al., 2003)

Fines Content, *FC*	Equation for Predicting V_S (m/s) [1]	Equation No.
< 40%	$V_S = 72.9\left(N^*_{60}\right)^{0.224} Z^{0.130} ASF$	Equation 12-2
10% to 35%	$V_S = 72.3\left(N^*_{60}\right)^{0.228} Z^{0.152} ASF$	Equation 12-3
< 10%	$V_S = 66.7\left(N^*_{60}\right)^{0.248} Z^{0.138} ASF$	Equation 12-4

[1] N^*_{60} = blows/0.3m = blows/ft (Section 7.8.1) and Z = depth in meters, ASF = Age Scaling Factors

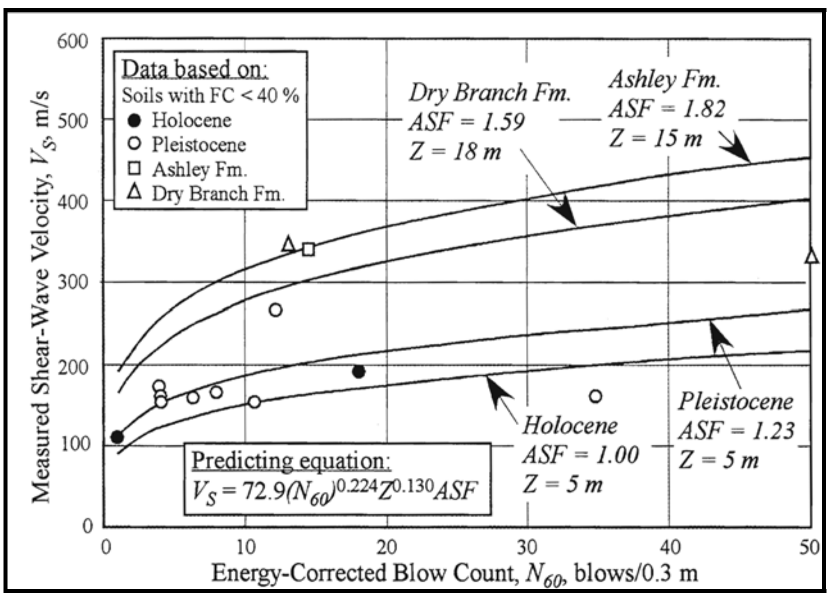


Figure 12-1, SPT (N_{60}) vs. Shear Wave (V_S) (Andrus et al., 2003)

Recommended age scaling factors (ASF) based on Andrus et al. (2003) are provided in Table 12-3.

Table 12-3, Recommended Age Scaling Factors (ASF) for SPT (Andrus et al., 2003)

Geologic Age and Location of Deposits	Fines Content [(1)], *FC* (%)	Age Scaling Factor, *ASF*	Database Range of Shear Wave Velocity, V_S	
			m/s	ft/s
Holocene SC Coastal Plain	< 40%	1.00	110 – 260	360 - 850
	10% to 35%	1.00	120 - 240	400 - 800
	< 10%	1.00	110 – 260	360 - 850
Pleistocene SC Coastal Plain	< 40%	1.23	150 – 270	500 - 900
	10% to 35%	1.08	160	550
	< 10%	1.28	150 – 270	500 - 900
Tertiary SC Coastal Plain Ashley Formation (Cooper Marl)	< 40%	1.82	340	1,100
	10% to 35%	1.71	340	1,100
Tertiary SC Coastal Plain Dry Branch Formation	< 40%	1.59	330 – 350	1,100 – 1,200
	10% to 35%	1.48	330 - 350	1,100 – 1,200

[(1)] *FC*= % passing #200 sieve

The procedures for using the V_s correlation equations in Table 12-2 are provided in Table 12-4.

Table 12-4, Procedure for Correlating SPT (N^*_{60}) to Shear Wave Velocity, V_s

Steps	Procedure Description
1	Perform a geotechnical subsurface exploration and identify subsurface soil geologic units, approximate age, and formation.
2	Determine fines content (FC) for soils at each SPT (N_{meas}) at depth (Z).
3	Compute standardized SPT blow count (N^*_{60}) to account for energy variations in SPT equipment. (Section 7.8.1)
4	Calculate shear wave velocity, V_S, for each (N^*_{60}) using Equation 12-2 and the appropriate *ASF* in Table 12-3. Equation 12-2 is the general equation used to estimate V_S for Sands with less than 40% fines content. If the fines content, *FC*, is known more definitive, then a better estimation can be made with Equations 12-3 and 12-4.
5	Plot a profile of calculated shear wave velocities, V_S, with respect to depth. If field shear wave velocity measurements have been made, plot this data on the profile and compare calculated shear wave results, V_S, with the measured V_S to verify appropriateness (accuracy) of SPT-V_S Equations.

12.3.2.2 CPT - Shear Wave Velocity, V_S, Estimation of SC Soils

Recommended equations to estimate shear wave velocities, V_S, for South Carolina soils are based on CPT tip resistance (q_c), depth (Z), soil behavior type (I_c), geologic age and location of deposit, and Age Scaling Factor (*ASF*). Equations for estimating shear wave velocities, V_S, of South Carolina soils are provided in Table 12-5. The CPT – V_S relationship for Holocene, Pleistocene, and Tertiary soils are plotted in Figures 12-2, 12-3, and 12-4, respectively.

Table 12-5, CPT (q_c) - Shear Wave Velocity, V_S, Equations for SC Soils (Andrus et al., 2003)

Soil Behavior Type, I_c	Equation for Predicting V_S (m/s) [(1)]	Equation No.
All Values	$V_S = 4.63 q_c^{0.342} I_c^{0.688} Z^{0.092} ASF$	Equation 12-5
< 2.05	$V_S = 8.27 q_c^{0285} I_c^{0.406} Z^{0.122} ASF$	Equation 12-6
> 2.60	$V_S = 0.208 q_c^{0.654} I_c^{1.910} Z^{-0.108} ASF$	Equation 12-7

(1) I_c = Soil Behavior Type Index (See Table 12-6)
(2) q_c = CPT tip resistance (kPa), Z = depth in meters, and ASF = Age Scaling Factors

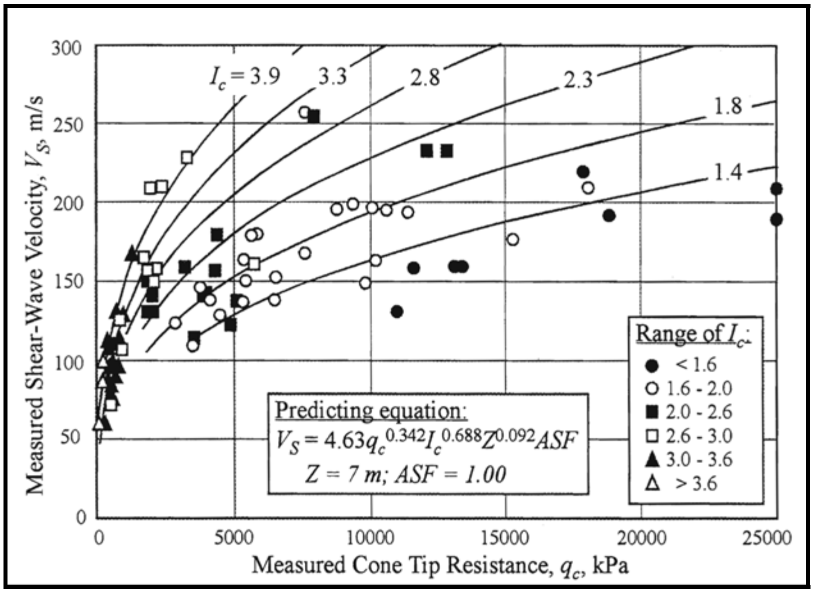


Figure 12-2, CPT – V_S Relationship for Holocene Soils (Andrus et al., 2003)

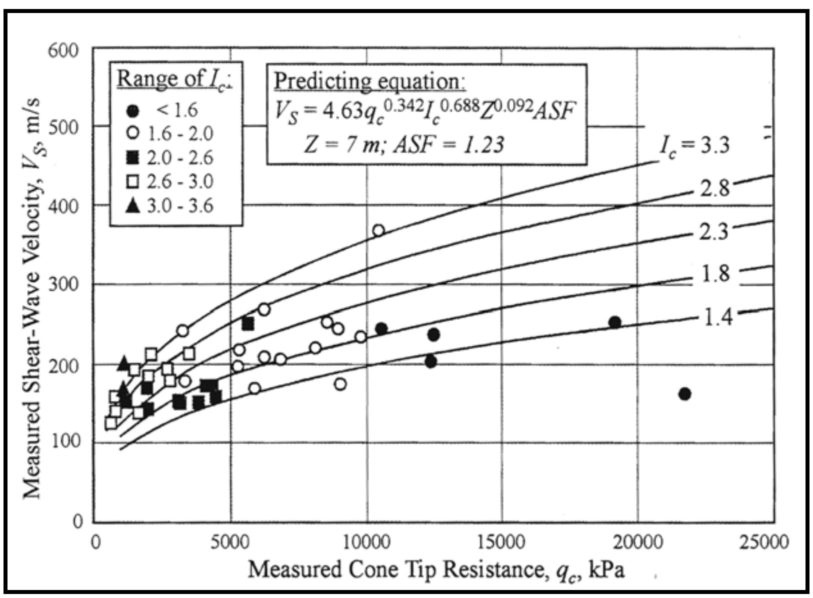


Figure 12-3, CPT – V_S Relationship for Pleistocene Soils (Andrus et al., 2003)

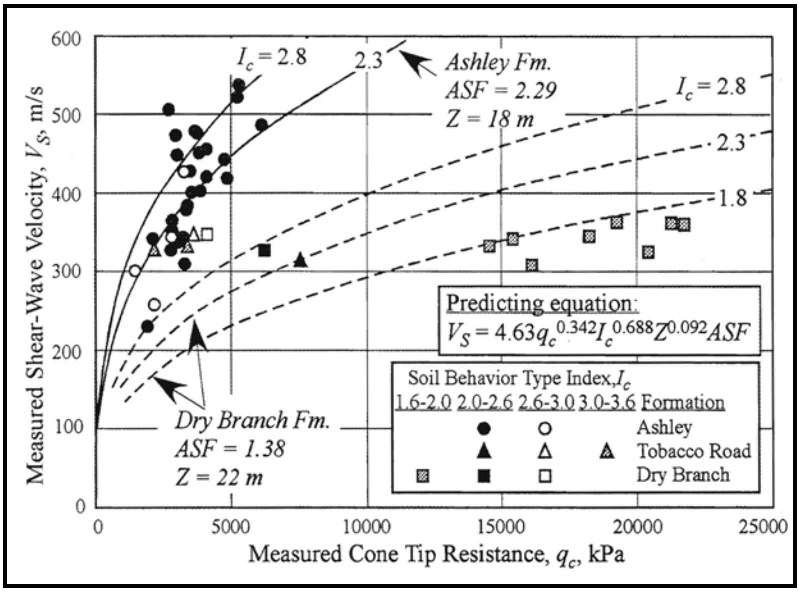


Figure 12-4, CPT – V_S Relationship for Tertiary Soils (Andrus et al., 2003)

Robertson (1990) established general soil behavior type, I_c, values as shown in Figure 12-5.

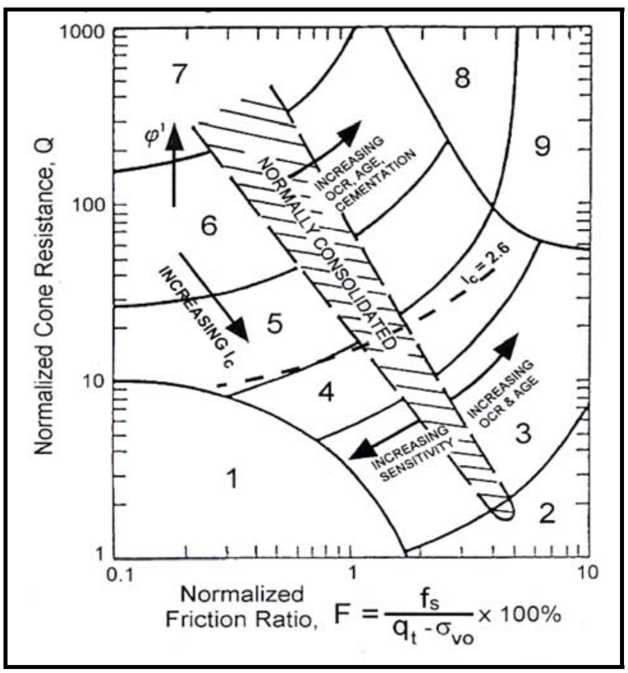


Figure 12-5, Normalized CPT Soil Behavior Type Chart (Robertson, 1990)

Table 12-6 indicates, for soil zones 1 thru 9 (shown in Figure 12-5), the soil behavior type description and the soil behavior index, I_c.

Table 12-6, Soil Behavior Type Index for CPT (Robertson, 1990)

Zone	Soil Behavior Type Description	Soil Behavior Type Index, I_c
1	Sensitive, Fine Grained	---
2	Organic Soils – Peat	$I_c > 3.60$
3	Clays – Silty Clay to Clay	$2.95 < I_c < 3.60$
4	Silt Mixtures – Clayey Silt to Silty Clay	$2.60 < I_c < 2.95$
5	Sand Mixtures – Silty Sand to Sandy Silt	$2.05 < I_c < 2.60$
6	Sands – Clean Sand to Silty Sand	$1.31 < I_c < 2.05$
7	Gravelly Sand to Sand	$I_c < 1.31$
8	Very Stiff Sand to Clayey Sand (1)	---
9	Very Stiff, Fine Grained (1)	---

(1) Heavily overconsolidated or cemented soils

The boundaries between soil zones 2 through 7 shown in Figure 12-5 can be differentiated by a soil behavior index, I_c.

$$I_c = \left[(3.47 - \log Q)^2 + (1.22 + \log F)^2\right]^{0.5}$$ Equation 12-8

The normalized cone resistance, *Q*, and the normalized friction ratio, *F*, are computed using the equations shown in Table 12-7.

Table 12-7, Normalized CPT Q and F Equations (Andrus et al., 2003)

Normalized CPT Value	Equation (1)	Equation No.
Normalized Cone Resistance, *Q*	$Q = \left[\frac{q_c - \sigma'_v}{P_a}\right]\left(\frac{P_a}{\sigma'_v}\right)^n$	Equation 12-9
Normalized Friction Ratio, *F*	$F = \left[\frac{f_s}{q_c - \sigma'_v}\right]100\%$	Equation 12-10

(1) q_c = CPT Tip Resistance (kPa); f_s = CPT Skin Resistance (kPa); P_a = Reference Stress = 100 kPa = 1 atm; σ'_V = Effective Vertical or Overburden Stress (kPa); n = exponent ranging from 0.5 to 1.0 (See Table 12-8)

The soil behavior index, I_c, is computed using Equations 12-8, 12-9, 12-10 and using an iterative procedure developed by Robertson and Wride (1998) as detailed in Table 12-8.

Table 12-8, Soil Behavior Index, I_c, Iterative Computational Procedure (Robertson and Wride, 1998)

1. Calculate soil behavior index, I_c, using n=1.0.
2. If soil behavior index, I_c, is > 2.60, use computed I_c using n=1.0
3. If soil behavior index, I_c, is < 2.60, recalculate I_c using n=0.50
   a. If the recalculated I_c is <2.60, use computed I_c using n=0.50
   b. If the recalculated I_c is >2.60, recalculate I_c using n=0.70

Recommended age scaling factors (*ASF*) based on Andrus et al., (2003) are provided in Table 12-9.

Table 12-9, Recommended Age Scaling Factors (ASF) for CPT (Andrus et al., 2003)

Geologic Age and Location of Deposits	Soil Behavior Description	Soil Behavior Type Index, I_c	Age Scaling Factor, *ASF*	Database Range of Shear Wave Velocity, v_S	
				m/s	ft/s
Holocene SC Coastal Plain	All Soils	All Values	1.00	60 – 260	200 - 850
	Clean Sand Silty Sand	< 2.05	1.00	110 – 260	350 - 850
	Clay, Silty Clayey Silt, Silty Clay	> 2.60	1.00	60 – 230	200 - 750
Pleistocene SC Coastal Plain	All Soils	All Values	1.23	130 – 300	450 – 1,000
	Clean Sand Silty Sand	< 2.05	1.34	160 – 300	500 – 1,000
	Clay, Silty Clayey Silt, Silty Clay	> 2.60	1.16	130 – 250	450 – 1,000
Tertiary SC Coastal Plain Ashley Formation (Cooper Marl)	All Soils	All Values	2.29	230 – 540	750 – 1,800
Tertiary SC Coastal Plain Tobacco Road Formation	All Soils	All Values	1.65	310 – 350	1,000 – 1,150
	Clay, Silty Clayey Silt, Silty Clay	> 2.60	1.42	330 – 350	1,100 – 1,150
Tertiary SC Coastal Plain Dry Branch Formation	All Soils	All Values	1.38	310 – 360	1,000 – 1,200
	Clean Sand, Silty Sand	< 2.05	1.33	310 – 360	1,000 – 1,200

The procedures for using the q_c correlation equations in Table 12-5 are provided in Table 12-10.

Table 12-10, Procedure for Correlating CPT (q_c) to Shear Wave Velocity, V_s

Step	Procedure Description
1	Perform a geotechnical subsurface exploration and identify subsurface soil geologic units, approximate age, and formation.
2	Calculate soil behavior index, I_c, for soils at each CPT (q_c) at depth (Z) using the Equations 12-8, 12-9, 12-10 and computational procedure listed in Table 12-8.
3	Convert CPT tip resistance, q_c, to kPa and depth, Z, in meters.
4	Calculate shear wave velocity, V_S, for each CPT tip resistance, q_c, value of interest using Equation 12-5 and the appropriate *ASF* value in Table 12-9. Equation 12-5 is the general equation used to estimate V_S for all values of I_c and values of $2.05 \le I_c \le 2.60$. A better estimation of shear wave velocity, V_S, can be obtained using Equations 12-6 for $I_c < 2.05$ or Equation 12-7 for $I_c > 2.60$. The *ASF* values in Table 12-9 listed for all values of I_c can be used with the general Equation 12-5. For a better estimation of *ASF*, the *ASF* values associated with soil with $I_c < 2.05$ or for $I_c > 2.60$ can also be used.
5	Plot profile of calculated shear wave velocities, V_S, with respect to depth. If field shear wave velocities measurements have been made, plot this data on the profile and compare calculated shear wave results, V_S, with the measured V_S to verify appropriateness (accuracy) of CPT-V_S Equations.

12.3.2.3 Corrected Shear Wave Velocity, V_{S1}, for Overburden Stress

Some analytical methods require that the shear wave velocity, V_S, be corrected for effects of effective overburden stress, σ'_V. Measured or calculated shear wave velocity, V_S, can be corrected for overburden stress using Equations 12-11 and 12-12.

$$V_{1,S} = V_S C_{VS} = V_S \left(\frac{P_a}{\sigma'_V} \right)^{0.25} \qquad \text{Equation 12-11}$$

$$C_{VS} = \left(\frac{P_a}{\sigma'_V} \right)^{0.25} \leq 1.4 \qquad \text{Equation 12-12}$$

Where effective overburden stress, σ'_V is in kPa and P_a is the reference stress of 100 kPa. The shear wave overburden correction, C_{VS}, is limited to 1.4. The P_a and σ'_V used to compute C_{VS} in Equation 12-12 must be in the same units.

12.3.3 South Carolina Reference Shear Wave Profiles

The shear wave profiles presented in this section are provided for reference purposes only. Project specific shear wave profiles should be developed from shear wave measurements as indicated in Chapter 4 and supplemented to deeper formations by the use of geologic publications, previous investigations, and reference shear wave profiles presented in this section.

A number of seismic studies have been performed in South Carolina that have yielded shear wave profiles for different parts of the state. The majority of the shear wave profiles in published references are in the Coastal Plain. Shear wave velocities were obtained by one of the following testing methods: Seismic Refraction, Seismic Reflection, Surface Wave (SASW and MASW), Downhole (including Seismic CPT), or Crosshole as described in Chapter 5. When shear wave measurements are not available for soil formations beyond the shear wave testing capabilities, estimates are typically made by using available shear wave data from formations previously tested or by using geologic information.

The shear wave velocity profile information contained in this section has been divided into three sections: USGS Shear Wave Velocity Data, SCEMD Seismic Risk and Vulnerability Study, and Published / SCDOT Shear Wave Velocity Profiles. A brief review of these reference shear wave velocity profiles is presented in the following sections.

12.3.3.1 USGS Shear Wave Velocity Data

The U.S. Geologic Survey (USGS) has compiled shear wave profiles in South Carolina in a report prepared by Odum et al. (2003). Shear wave measurements were obtained by seismic refraction/reflection profiling techniques for nine locations in South Carolina as indicated in Figure 12-6 and listed below:

1. Lake Murray Dam Spillway, Columbia, SC: Paleozoic Rocks of the Carolina Slate Group.

2. Fort Jackson Military Base, Columbia, SC: Cretaceous Tuscaloosa Formation (Middendorf Formation)

3. Deep Creek School: Peedee Formation (Upper Cretaceous)

4. Black Mingo: Black Mingo Formation (lower Eocene-Wilcox Group)

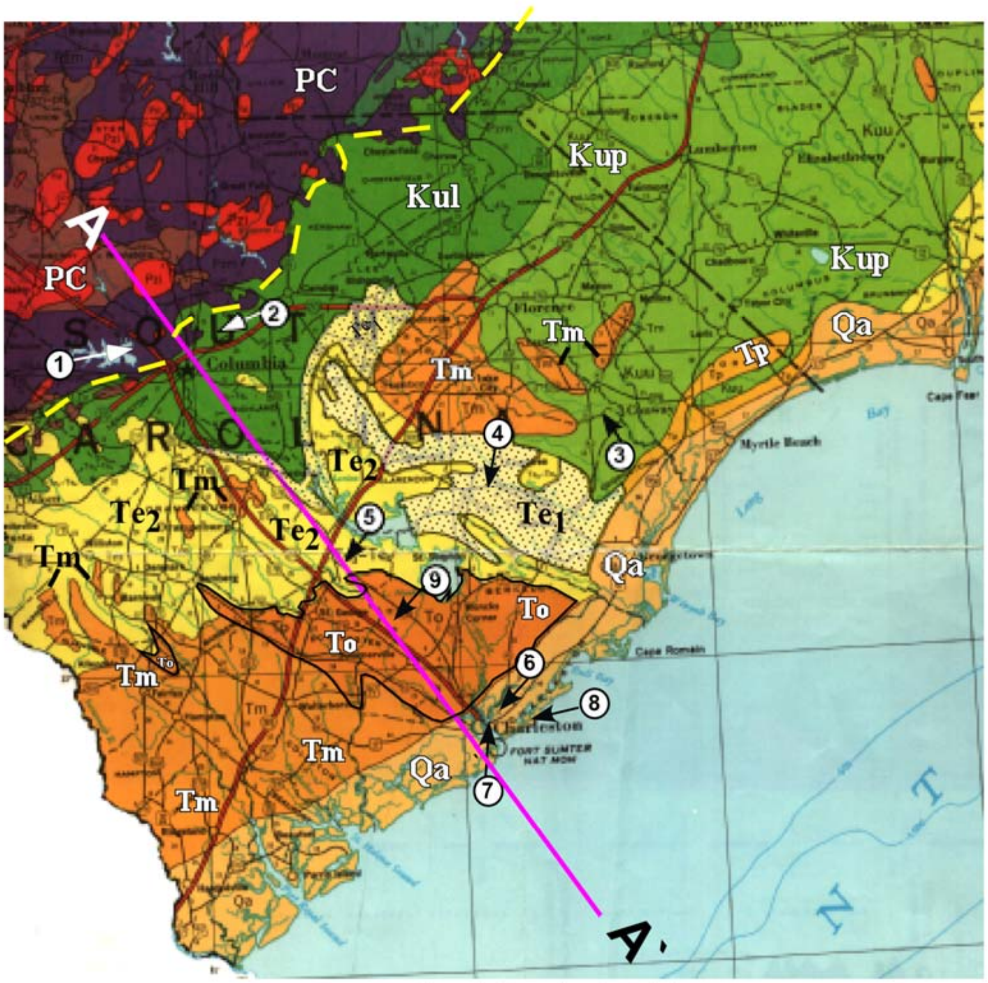
5. Santee Limestone: Santee Limestone (Middle Eocene-Clayborne Group)

6. The Citadel, Charleston, SC: Quaternary deposits (barrier sand facies) overlying Upper Tertiary Cooper Group (Ashley and Parkers Ferry Formations) - The Citadel

7. Highway US 17 Overpass next to Ashley River Memorial Bridge: Quaternary deposits overlying Upper Tertiary Cooper Group (Ashley and Parkers Ferry Formations)

8. Isle of Palms, Charleston, SC: Quaternary deposits (beach and barrier-island sand facies) overlying Upper Tertiary Cooper Group (Ashley and Parkers Ferry Formations)

9. U.S. National Seismograph Network (USNSN) installation site: Quaternary deposits overlying Upper Tertiary Cooper Group (Ashley and Parkers Ferry Formations)



Generalized Geologic Units

Qa	Surface soils and fill, Pleistocene and Holocene of alluvial and marine origin
Tp	Pliocene sandy shell marl
Tm	Miocene sandy phosphatic marl, shaley (Hawthorn Group)
To	Oligocene massive phosphatic marl (Cooper Group-Ashley Fm.)
Te_2	Eocene upper units- calcilutite and limestones (Cooper Gp - Parkers Ferry Fm.) Santee Ls.
	Eocene lower units-clayey to sandy siltstone and limestone (Wilcox Group, Black Mingo Fm.)
Kup	Cretaceous- calcareous clayey sands and limestone (Peedee Fm.)
Kul	Cretaceous- predominantly clean to silty sands and pure clay (Tuscaloosa Fm.)
Pc	Precambrian- metamorphosed early Paleozoic and older sedimentary and volcanic rocks and granitic intrusions (Carolina Slate Group)

Figure 12-6, USGS Nine Study Locations (Odum et al., 2003)

Shear wave (V_S) profiles for the nine USGS sites are summarized in Table 12-11 and shown in Figure 12-7.

Table 12-11, USGS Shear Wave Profile Summary (Odum et al., 2003)

Site No.	Site Name	Latitude (degrees)	Longitude (degrees)	Surficial Geology [1]	Highest V_S in Upper 164' (50 m) (m/s)	Highest V_S in Upper 164' (50 m) (ft/sec)	Description [1]
1	Lake Murray Spillway	35.052	81.210	Fill, Pz	2,674 @ 23 m	8,770 @ 75 ft	Carolina Slate Group (P_z)
2	Fort Jackson	34.028	90.912	K_u	866 @ 27 m	2,840 @ 89 ft	Tuscaloosa Fm
3	Deep Creek School	33.699	79.351	Q?, K_u	710 @ 22 m	2,330 @ 72 ft	Q over Peedee Fm
4	Black Mingo	33.551	79.933	Q, T_l	855 @ 9 m	2,805 @ 30 ft	Q over Eocene Wilcox Group
5	Santee Ls	33.235	80.433	T_l	932 @ 7 m	3,057 @ 23 ft	Santee Limestone
6	The Citadel, Charleston	32.798	79.958	Q, T_u	795 @ 78 m	2,608 @ 256 ft	Q over T_u (Cooper Group)
7	US Hwy. 17, Charleston	32.785	79.955	Fill, Q	247 @ 11 m	810 @ 36 ft	Q over T_u (Cooper Group)
8	Isle of Palms	32.795	79.775	Q_h, T_u	497 @ 23 m	1,630 @ 75 ft	Q over T_u (Cooper Group)
9	USNSN	33.106	80.178	Q, T_u	792 @ 10 m	2,598 @ 33 ft	Q over T_u (Cooper Group)

[1] Definitions: Q – Quartenary; T_u – upper Tertiary; T_l – lower Tertiary; K_u – upper Cretaceous; P_z - Paleozoic

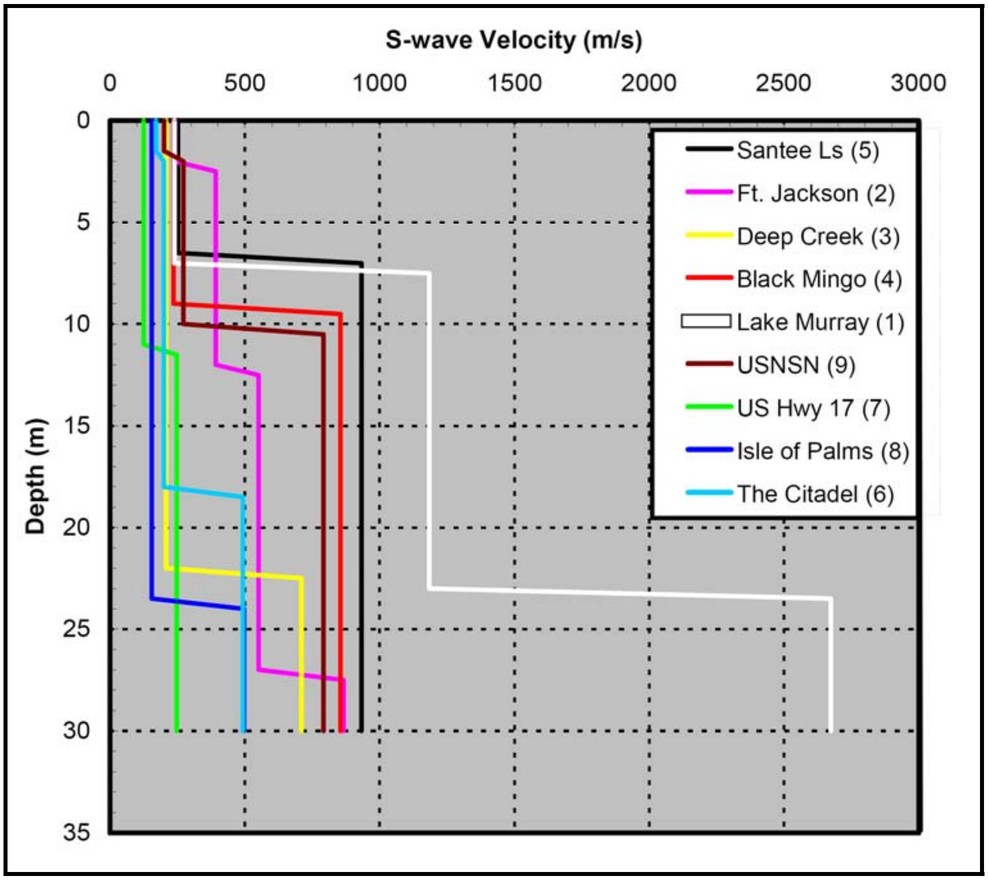


Figure 12-7, USGS Shear Wave V_S Profile (Odum et al., 2003)

The shear wave (V_S) and compression wave (V_P) profiles developed for the nine sites are shown in Figures 12-8 and 12-9. The columns show successively higher velocity layers V1, V2, and V3, indicated by yellow, blue, and light brown, respectively. For a detailed interpretation of the results shown in these profiles refer to Odum et al. (2003).

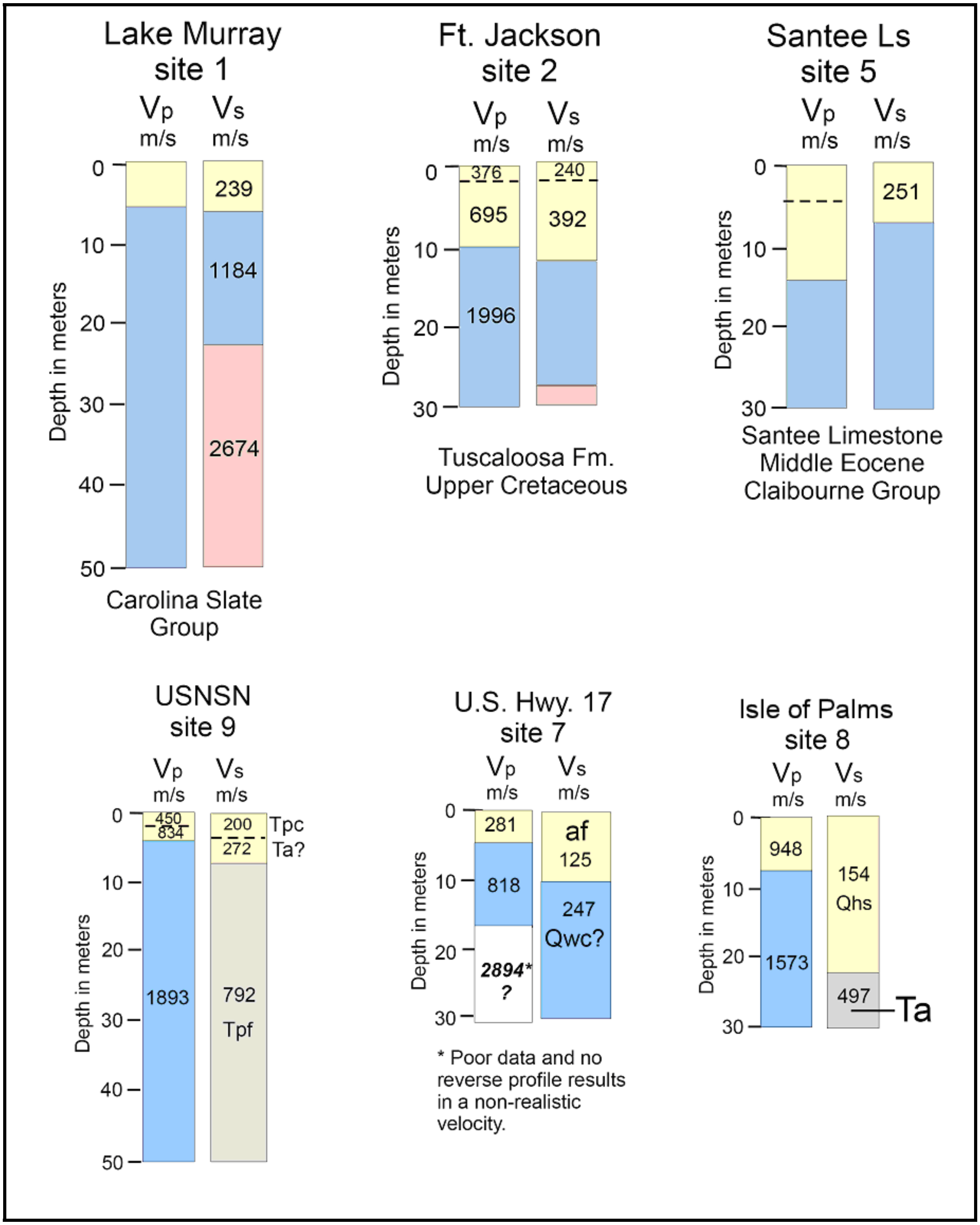


Figure 12-8, USGS Sites 1, 2, 5, 9, 7, and 8
(Odum et al., 2003)

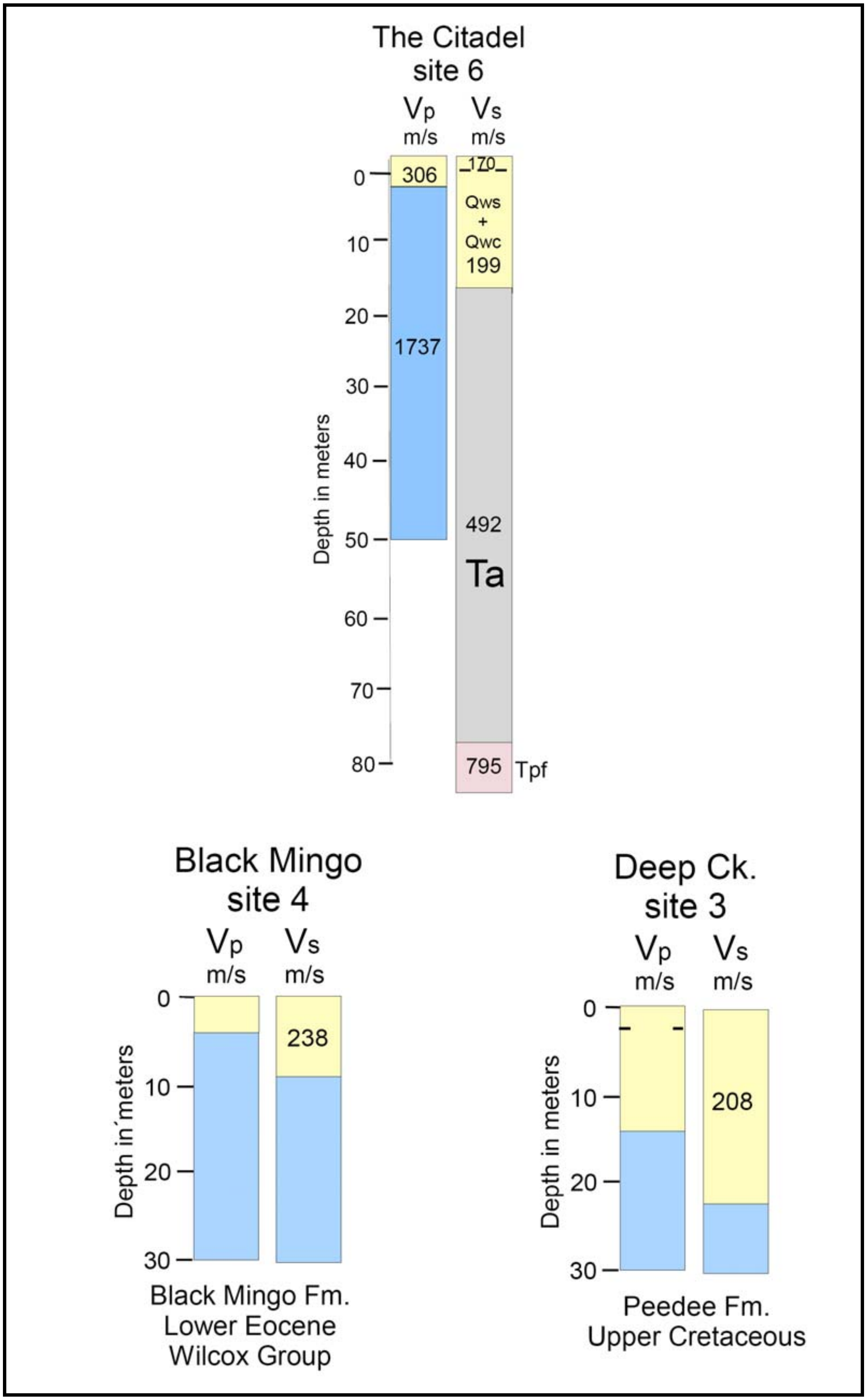


Figure 12-9, USGS Sites 6, 4, 3
(Odum et al., 2003)

12.3.3.2 SCEMD Seismic Risk and Vulnerability Study

A study was prepared by URS Corporation (2001) for the South Carolina Emergency Management Division (SCEMD). This study evaluated the potential losses resulting from four scenario earthquakes that may occur in South Carolina sometime in the future. South Carolina was divided into four site response categories based on physiographic provinces, surficial geology, and trends in subsurface data. The four site categories that were selected for this study are: Piedmont, Savannah River, Charleston, and Myrtle Beach. The extent of these site response categories are shown on a South Carolina map in Figure 12-10. The shear wave profiles for the Piedmont, Savannah River, Charleston, and Myrtle Beach are shown in Figures 12-11, 12-12, 12-13, and 12-14, respectively. For a detailed explanation of the base shear wave profiles used in this study refer to SCEMD report prepared by URS Corporation (2001).

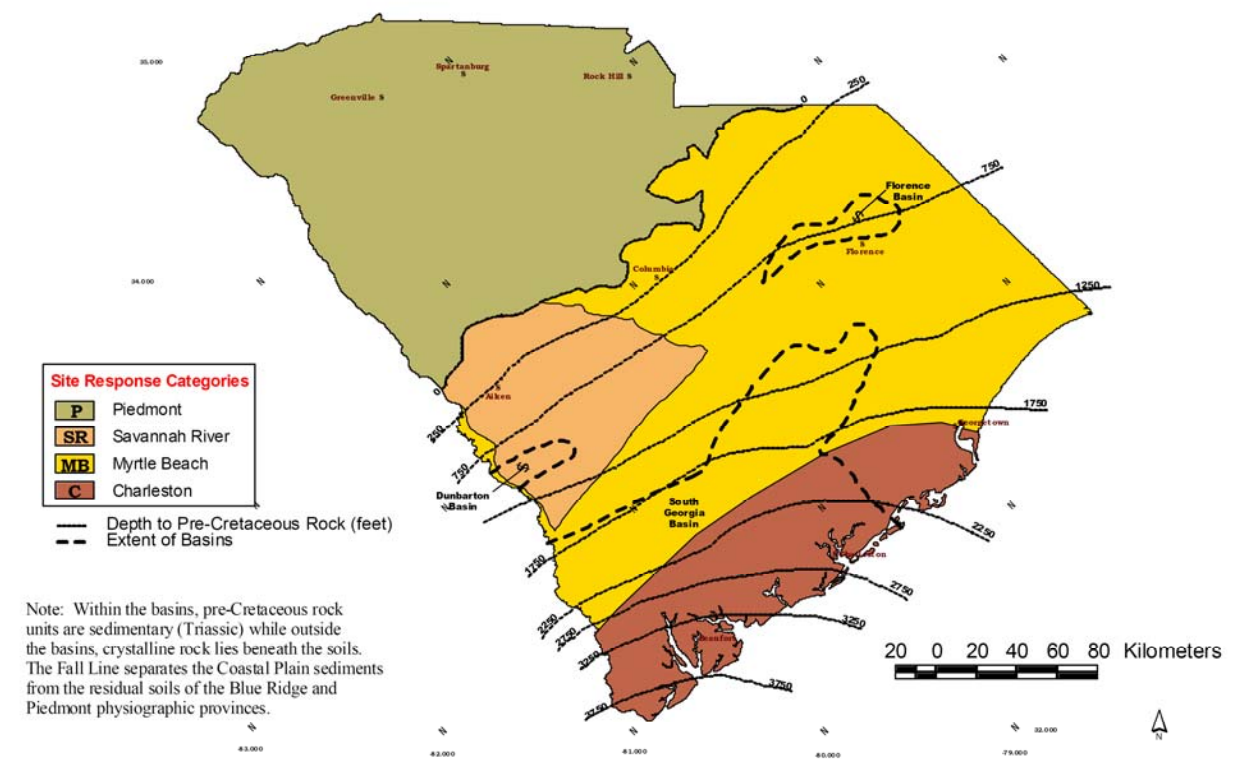


Figure 3-5. Site Response categories and depth to pre-Cretaceous rock.

Figure 12-10, Site Response Categories and Depth To Pre-Cretaceous Rock (URS Corporation, 2001)

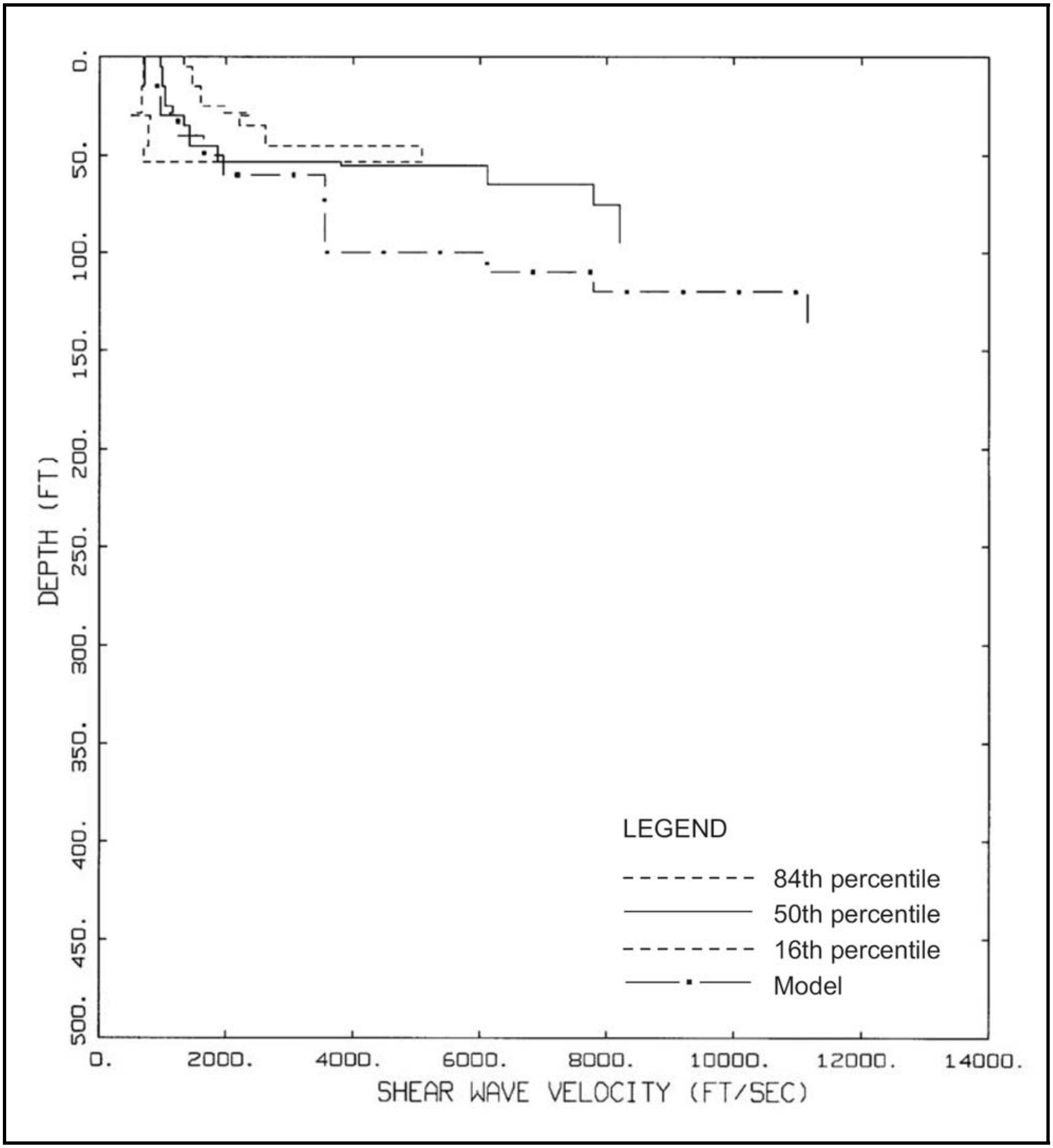


Figure 12-11, Piedmont/Blue Ridge Site Response Category Base Vs Profile (URS Corporation, 2001)

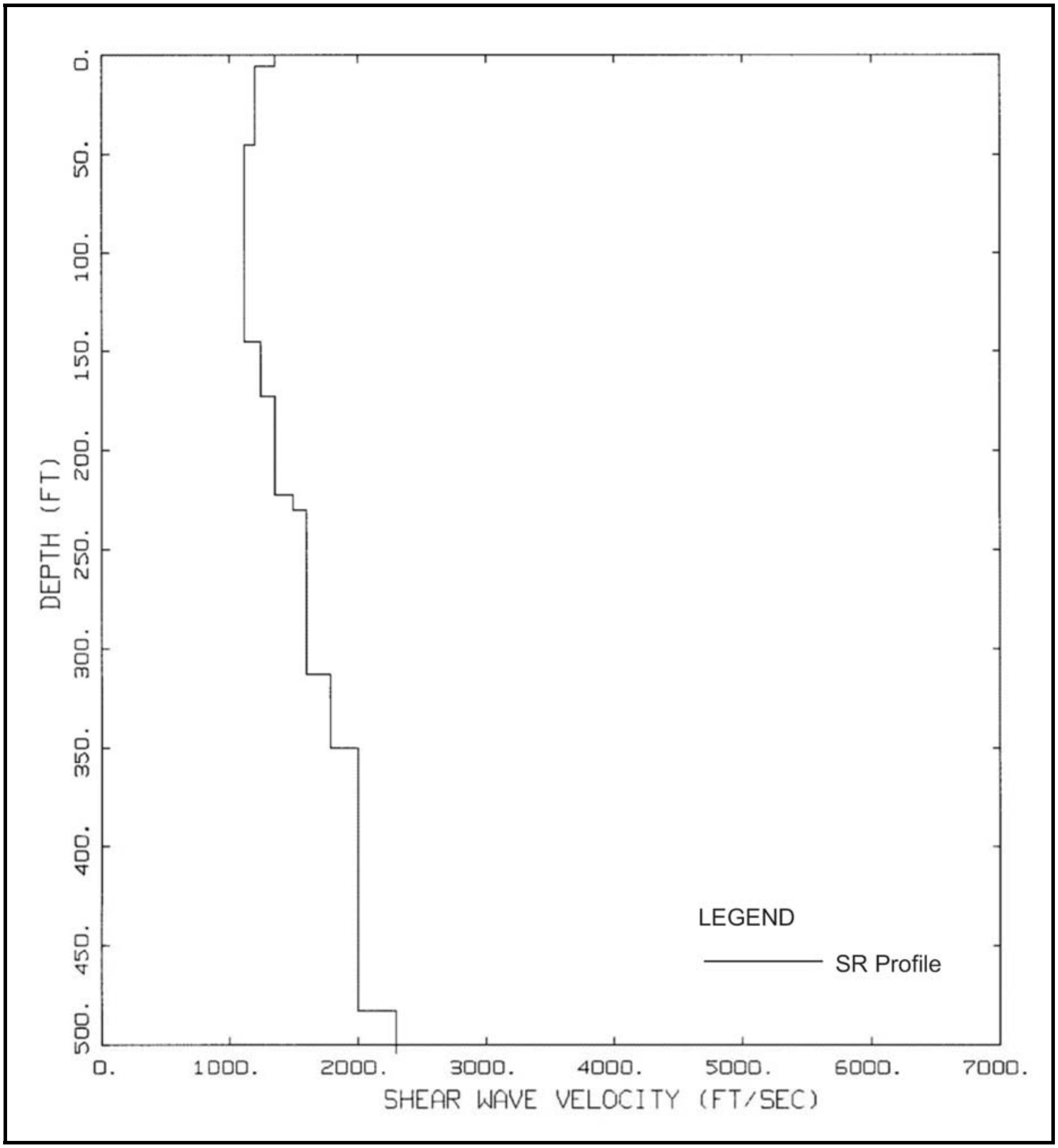


Figure 12-12, Savannah River Site Response Category Base Vs Profile (URS Corporation, 2001)

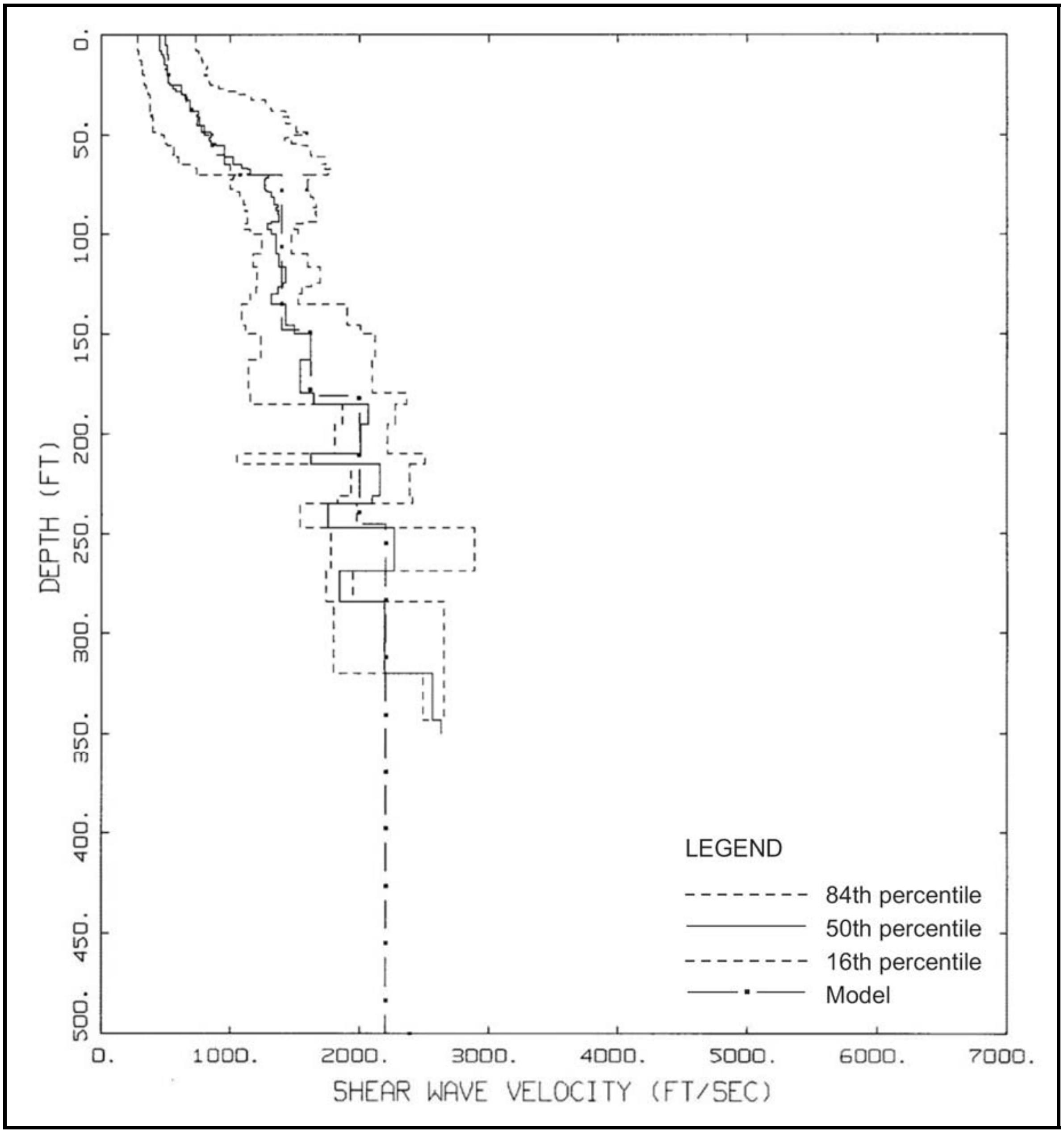


Figure 12-13, Charleston Site Response Category Base Vs Profile (URS Corporation, 2001)

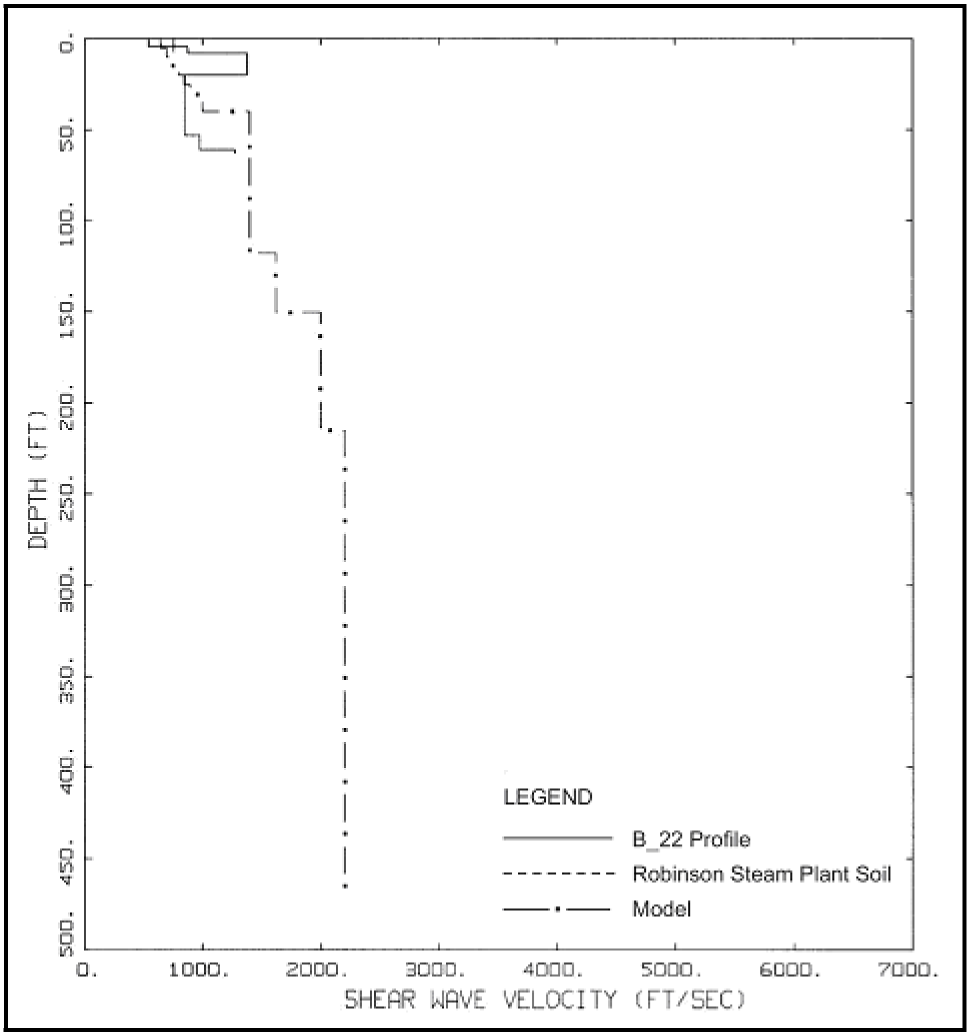


Figure 12-14, Myrtle Beach Site Response Category Base Vs Profile (URS Corporation, 2001)

12.3.3.3 Published / SCDOT Shear Wave Velocity Profiles

A review of published shear wave velocity profiles has been compiled to provide additional reference data for use in characterizing sites in South Carolina. The shear wave profiles are provided as references. For a detailed description of the geologic formation and geotechnical investigation, refer to the source documents. The list of the shear wave profiles compiled is provided below:

1. Seismic CPT and Geophysical shear wave profiles taken in Piedmont soils from the National Geotechnical Experimentation Sites (NGES) located at Opelika, Alabama. The Seismic CPT is shown in Figure 12-15 and the geophysical testing is shown in Figure 12-16. This site is generally accepted to be representative of Piedmont surface soils.

2. Seismic CPT shear wave profile taken at the Savannah River site in South Carolina is shown in Figure 12-17. This shear wave profile is generally representative of the soils at the U.S. Department of Energy Savannah River Site.

3. Seismic CPT shear wave profile taken at the Ravenel Bridge (Cooper River Bridge), located in Charleston, South Carolina, is shown in Figure 12-18.

4. Seismic CPT shear wave profiles were taken at Wetland Bridges 1 and 3 on US 17 between US Highway 21 intersection in Gardens Corner and the Combahee River. Two shear wave profiles were developed for Bridges 1 & 2 and Bridges 3 & 4 as shown in Figure 12-19. The SCPT B-14 taken at Bridge 1 is shown in Figure 12-20 and B-5A taken at Bridge 3 is shown in Figure 12-21.

5. Seismic CPT shear wave profiles were taken for a new bridge on US 378 over Great Pee Dee River, approximately 18 miles east of Lake City, South Carolina. Representative shear wave profiles from two SCPT SC3 and SC4 are shown in Figure 12-22 and 12-24, respectively. The corresponding SCPT logs for SC3 and SC4 are shown in Figures 12-23 and 12-25, respectively.

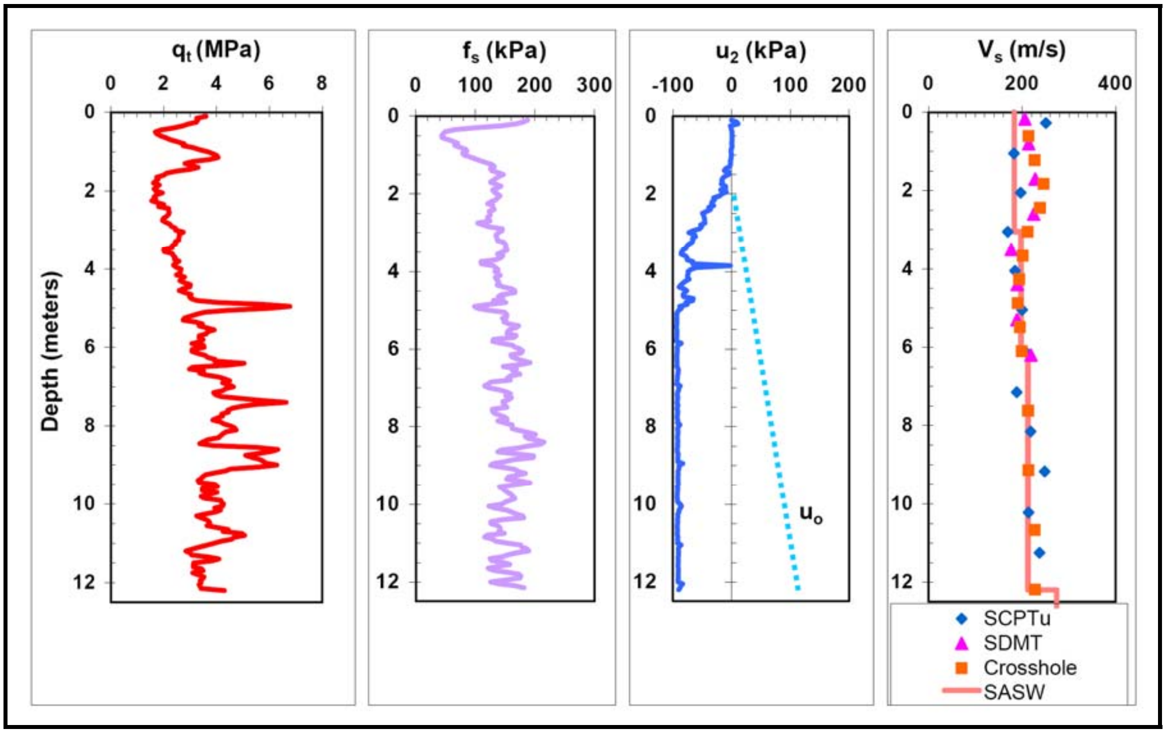


Figure 12-15, SCPT Piedmont Profile - NGES Opelika, Alabama (Mayne et al., 2000)

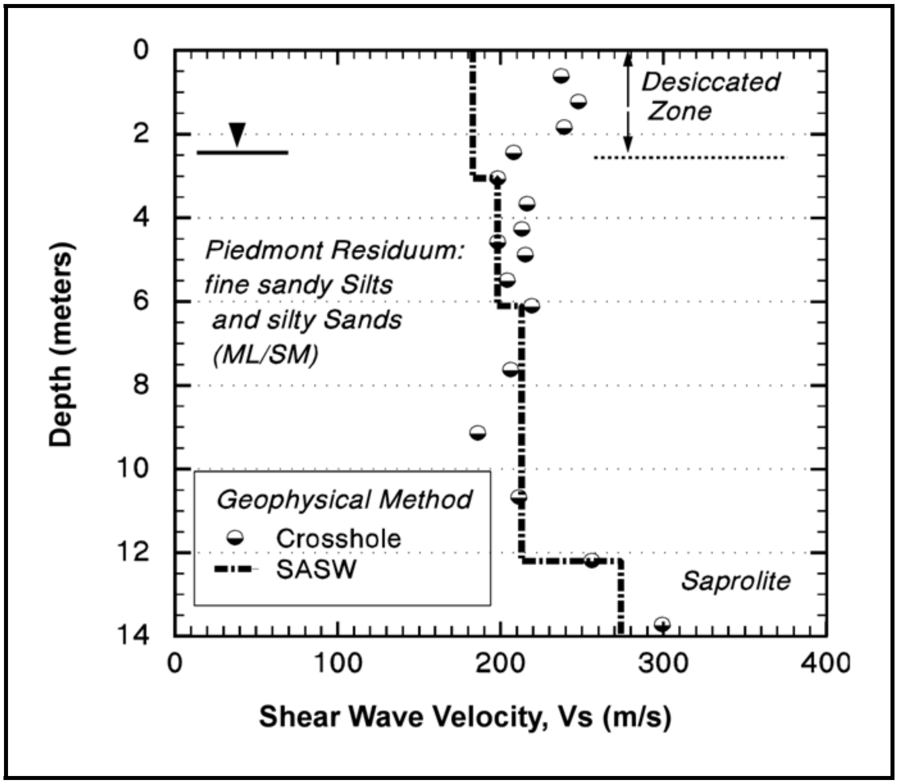


Figure 12-16, Geophysical V_S Piedmont Profile - NGES Opelika, Alabama (Mayne et al., 2000)

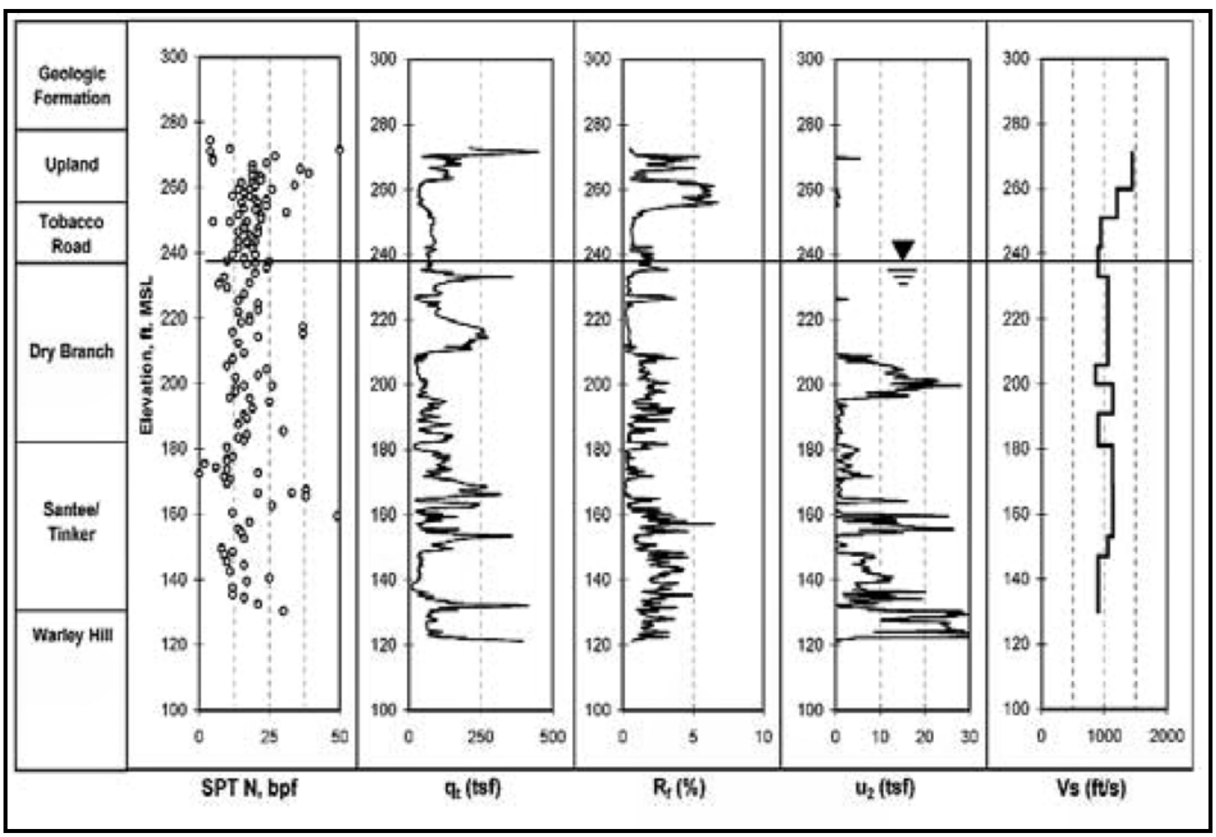


Figure 12-17, SCPT Profile Savannah River, South Carolina (Lewis et al., 2004)

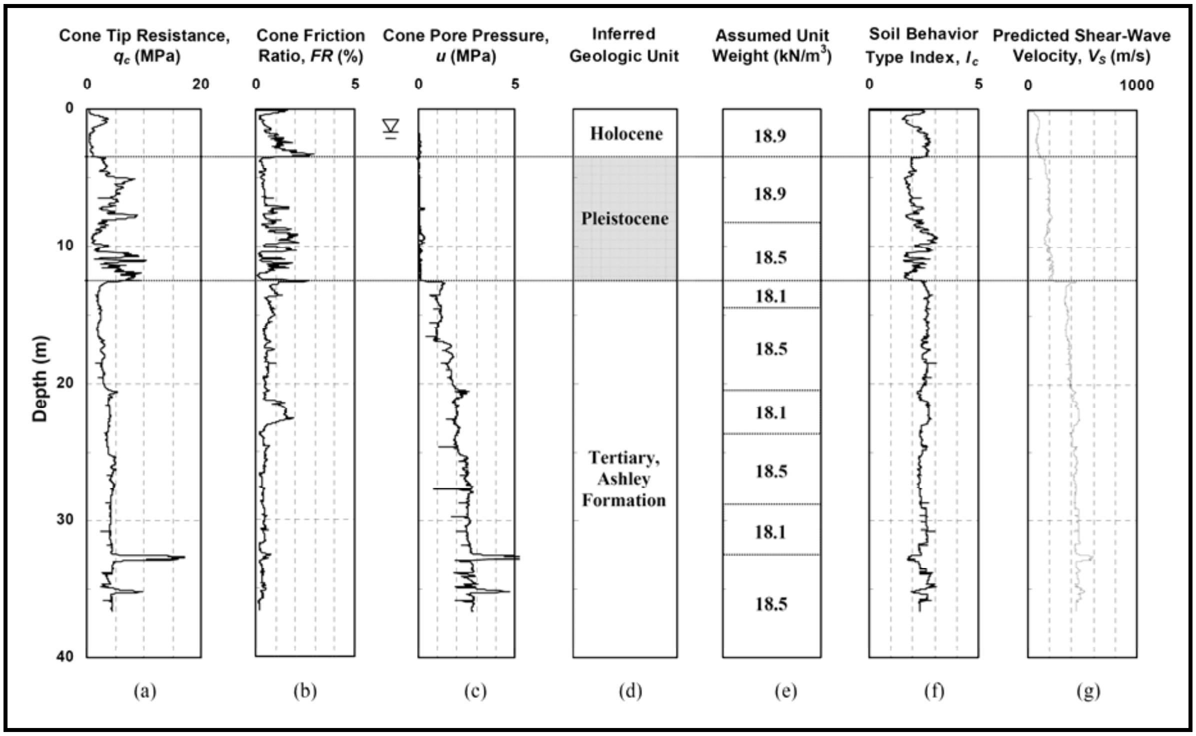


Figure 12-18, SCPT Profile (DS-1) Cooper River Bridge, Charleston, SC (S&ME, 2000)

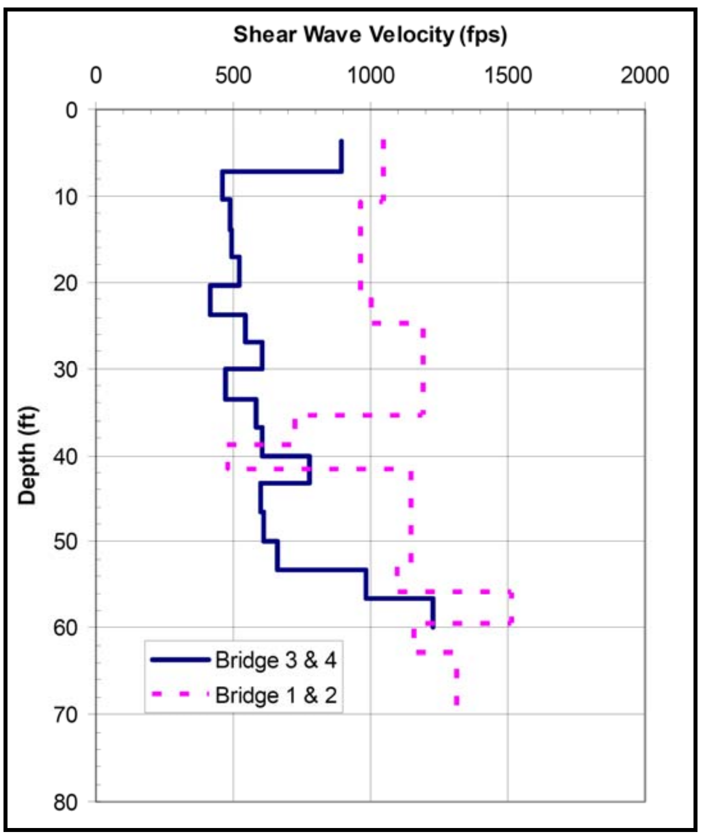


Figure 12-19, Shear Wave Profile US 17, Beaufort County, South Carolina (S&ME, 2007)

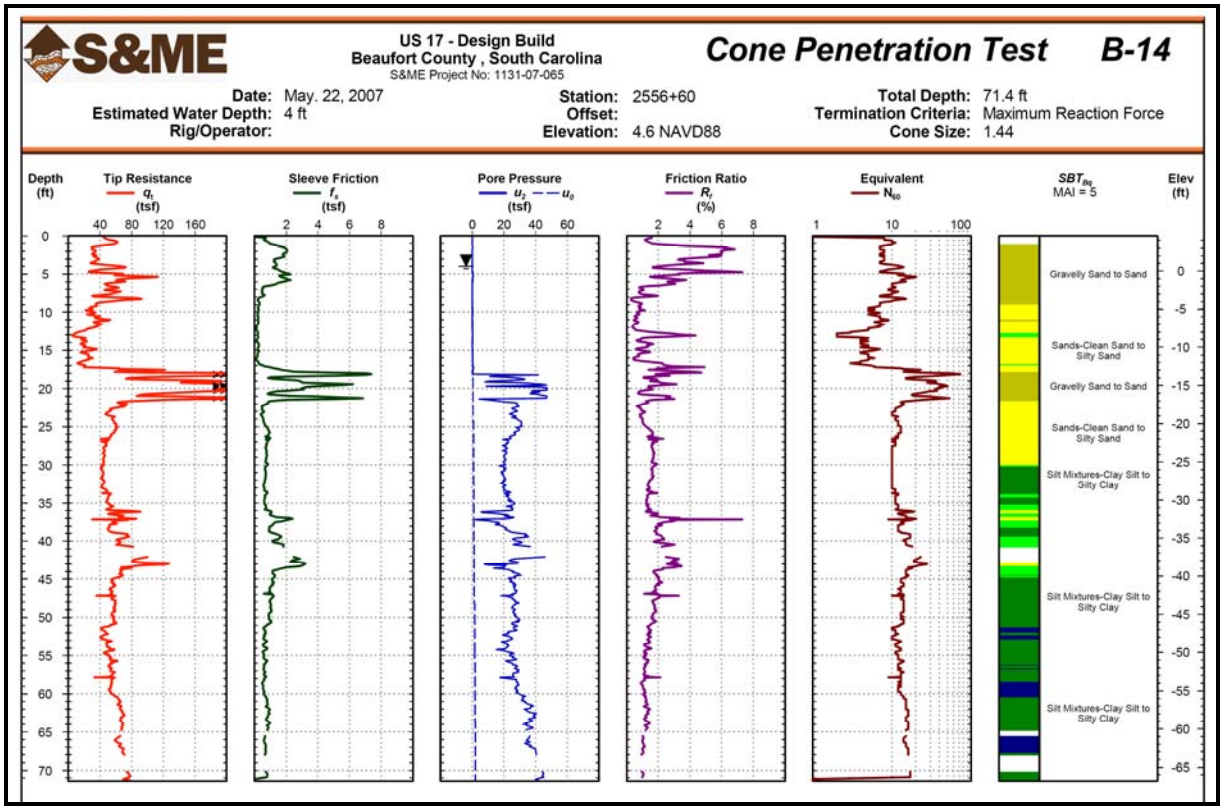


Figure 12-20, SCPT (B-14) US 17 Bridge 1, Beaufort County, South Carolina (S&ME, 2007)

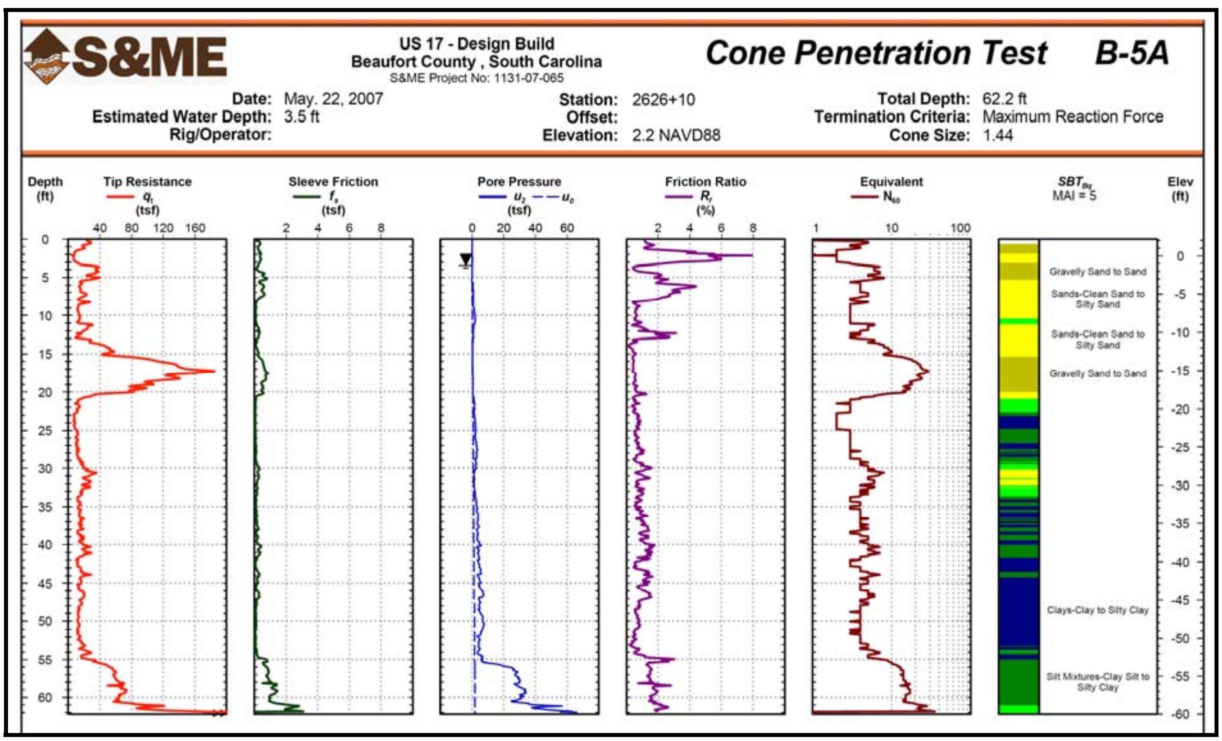
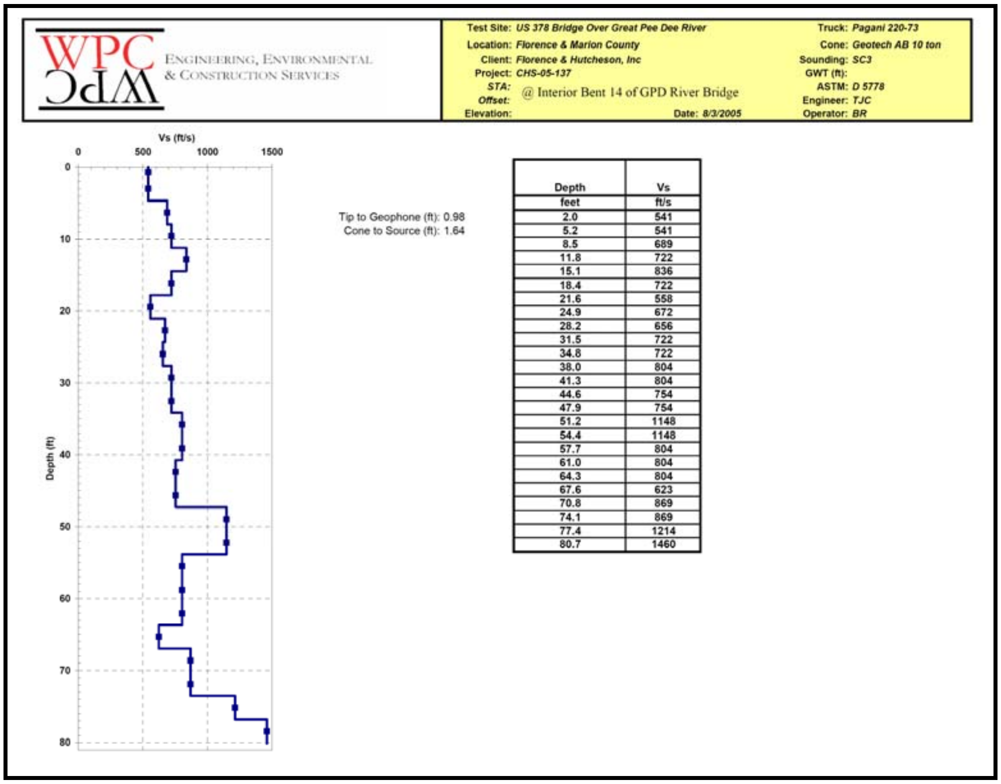


Figure 12-21, SCPT (B-5A) US 17 Bridge 3, Beaufort County, South Carolina (S&ME, 2007)



Depth	Vs
feet	ft/s
2.0	541
5.2	541
8.5	689
11.8	722
15.1	836
18.4	722
21.6	558
24.9	672
28.2	656
31.5	722
34.8	722
38.0	804
41.3	804
44.6	754
47.9	754
51.2	1148
54.4	1148
57.7	804
61.0	804
64.3	804
67.6	623
70.8	869
74.1	869
77.4	1214
80.7	1460

Figure 12-22, Shear Wave Profile (SC3) - US 378, Lake City, South Carolina (Florence & Hutcheson, 2006)

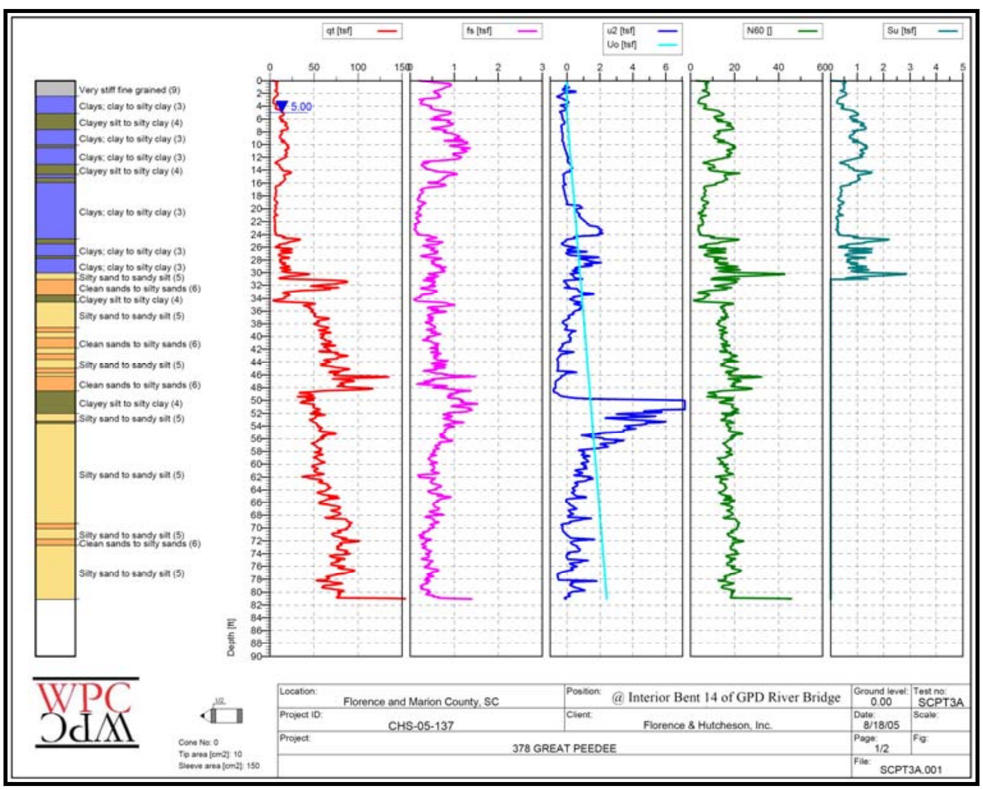
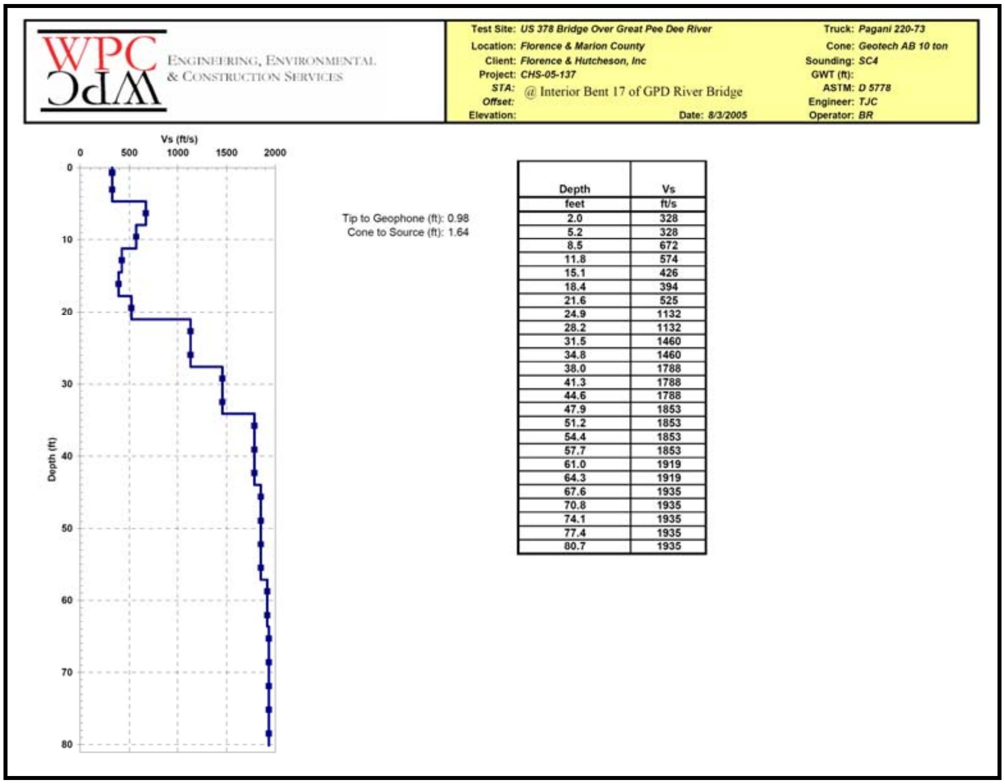


Figure 12-23, SCPT (SC3) - US 378, Lake City, South Carolina (Florence & Hutcheson, 2006)



Depth	Vs
feet	ft/s
2.0	328
5.2	328
8.5	672
11.8	574
15.1	426
18.4	394
21.6	525
24.9	1132
28.2	1132
31.5	1460
34.8	1460
38.0	1788
41.3	1788
44.6	1788
47.9	1853
51.2	1853
54.4	1853
57.7	1853
61.0	1919
64.3	1919
67.6	1935
70.8	1935
74.1	1935
77.4	1935
80.7	1935

Figure 12-24, Shear Wave Profile (SC4) - US 378, Lake City, South (Florence & Hutcheson, 2006)

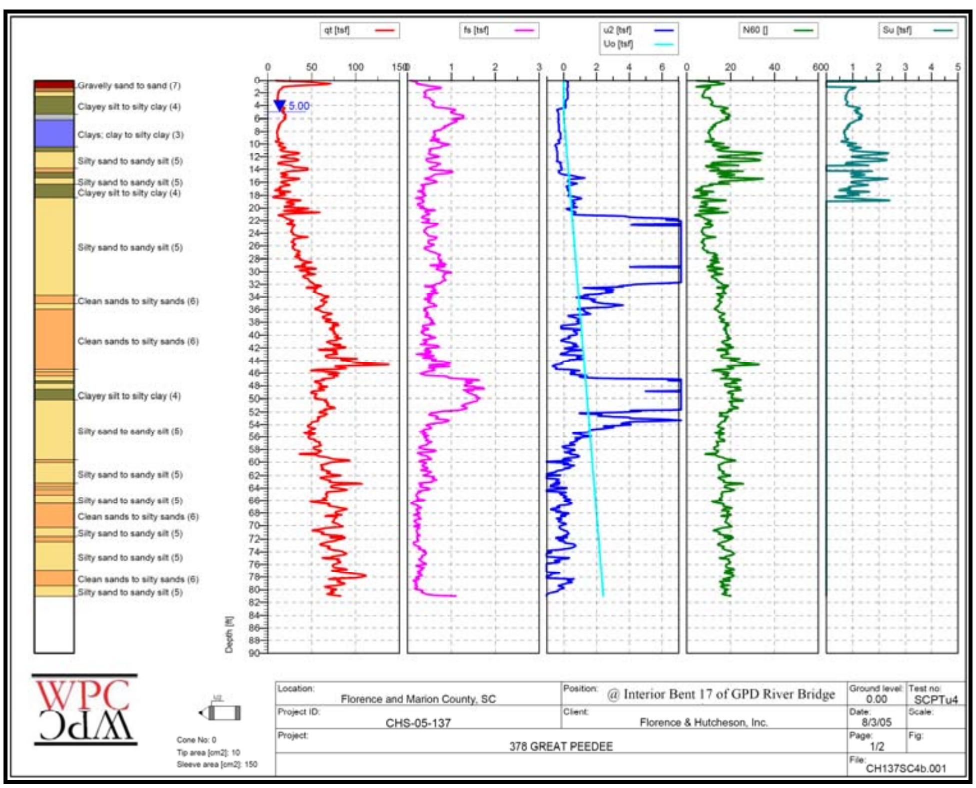


Figure 12-25, SCPT (SC4) - US 378, Lake City, South Carolina (Florence & Hutcheson, 2006)

12.3.4 Site Stiffness

Site stiffness is a measure of the overall soil stiffness (Section 12.3.2) of the soil layers to a specific depth of interest. Site stiffness in this Manual is computed as the weighted average of the shear wave velocities over a prescribed depth of the soil profile. The shear wave velocities, V_S, are not corrected for overburden. The weighted average can be computed by either using measured shear wave velocities obtained during the geotechnical site investigation or by using correlated shear wave velocities obtained from in-situ tests such as the Standard Penetration Test (SPT) and Cone Penetrometer Test (CPT) as indicated in previous sections.

Site stiffness in ft/sec (m/s) can be computed from measured shear wave velocities as indicated in the following equation.

$$\textit{Site Stiffness} = \frac{d_T}{t_d} \qquad \text{Equation 12-13}$$

Where,

d_T = total depth where shear wave velocities are being averaged in feet (m)

t_d = time that it takes for the shear wave to travel from the d_T to the ground surface (seconds)

Site stiffness in ft/sec (m/s) can also be computed by

$$\textit{Site Stiffness} = \frac{d_T}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}} \qquad \text{Equation 12-14}$$

Where,

d_T = total depth where shear wave velocities are being averaged in feet (m)

V_{si} = shear wave velocity of layer *i* in ft/sec (m/s)

d_i = thickness of any layer *i* between 0 and d_t

The distance below the ground surface, d_T, where the weighted shear wave velocities are computed is dependent on the type of geotechnical earthquake engineering analysis being performed. Consequently, site stiffnesses are designated and defined differently based on the depth of the zone of influence that shear wave velocity has on the computations that are being performed. The criteria for computing site stiffness for different types of geotechnical engineering correlations are provided in Table 12-12.

Table 12-12, Site Stiffness Definitions

Geotechnical Earthquake Engineering Correlation	Section Referenced	Site Stiffness Designation	d_T
Site Class Determination	12.4	$\bar{V}_s$	100 ft (30 m) below Z_{DTM} (1)
Nonlinear shear mass participation factor (r_d).	13.10.1	$V^*_{S,40'}$	40 feet (12 m)

(1) Z_{DTM} = Depth-to-motion. Additional guidance in determining d_T is provided in Sections 12.4.1 and 12.4.2.

12.3.5 Equivalent Uniform Soil Profile Period and Stiffness

The equivalent uniform soil profile period, T^*, and equivalent uniform soil profile stiffness, $V^*_{S,H}$, are used to compute the natural period of the site, T_N, as indicated in Section 12.6. The thickness of the profile (*H*) begins at the depth where the ground motion is of interest to the structure being designed (See Depth-to-Motion, Section 12.4.2) and extends to the depth where the motion is being generated, typically either the B-C boundary or a hard-rock basement outcrop (see Chapter 11). A comprehensive evaluation of how to determine the fundamental period of the soil profile has been made by Dobry et al. (1976). A simple and accurate method to determine the fundamental period of the soil profile is the Successive Two Layer Approach proposed by Madera (1970).

The Successive Two Layer Approach consists of solving for the fundamental period of two soil layers at a time, and then repeating the procedure successively (from the top to bottom of profile) until the entire soil profile is modeled as a single equivalent layer having a fundamental period, T^*. The Successive Two Layer Approach to compute the equivalent uniform soil profile period, T^*, and stiffness, $V^*_{S,H}$, is provided in Table 12-13.

Table 12-13, Successive Two Layer Approach

Step	Procedure Description
1	Begin with the layer at the top (n=1) of the profile under evaluation and continue working to the bottom of the profile. Compute the periods, T_A and T_B for top soil layers *A* (n) and bottom soil layer *B* (n + 1) using the following equations: $$T_A = \frac{4H_A}{V_{SA}}$$ Equation 12-15 $$T_B = \frac{4H_B}{V_{SB}}$$ Equation 12-16 Where, H_A = thickness of layer *A* in feet (m) H_B = thickness of layer *B* in feet (m) V_{SA} = shear wave velocity of layer *A* in ft/sec (m/s) V_{SB} = shear wave velocity of layer *B* in ft/sec (m/s) n = soil layer number (where top layer n = 1)
2	Compute the ratio of H_A/H_B and T_B/T_A.
3	Obtain the ratio of the uniform period, *T*, to T_A (T/T_A) for the combined two-layer system using Figure 12-26. Where T is the fundamental period for the two-layer
4	Compute the fundamental period, T, of the two layer system (A + B) using the following equation. $$T = \left(\frac{T}{T_A}\right) T_A$$ Equation 12-17 Where, $\left(\frac{T}{T_A}\right)$ = Ratio obtained from Figure 12-26 T_A = fundamental period of layer *A* in seconds (Equation 12-15)
5	Repeat items 1 through 4, where the combined two-layer system from step 4 becomes layer A with a fundamental period, $T_A = T$. Continue successively until the entire soil profile has been evaluated and there is a single fundamental period, *T*, for the entire soil profile. At this time, the single fundamental period, *T*, for the entire soil profile becomes equal to the equivalent uniform soil profile period, T^*.
6	Compute the equivalent uniform soil profile stiffness, $V^*_{S,H}$, from the following equation. $$V^*_{S,H} = \frac{4H}{T^*}$$ Equation 12-18 Where, *H* = thickness of the entire soil profile layer in feet (m) T^* = equivalent uniform soil profile period in seconds (Step 5)

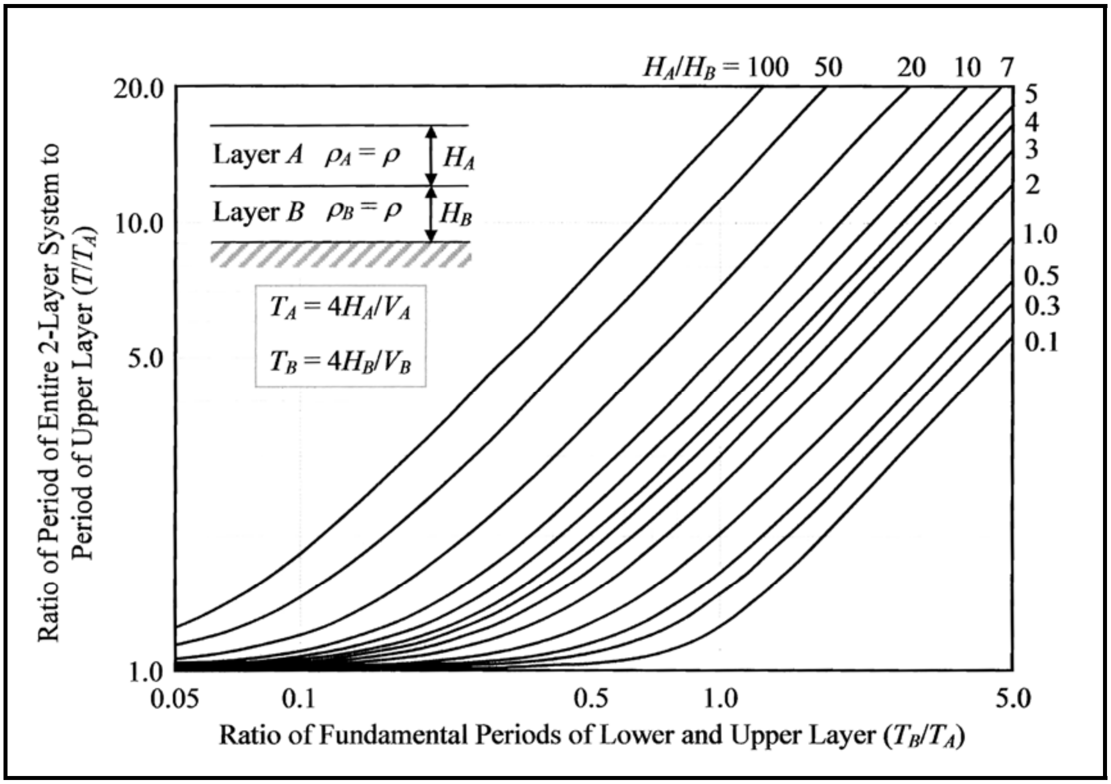


Figure 12-26, Fundamental Period of Two-Layer System
(Oweis et al., 1975, Adapted by Green (2001))

12.3.6 Shear Modulus Reduction Curves

Shear modulus reduction curves are typically presented as normalized shear modulus, G/G_{max} vs. shear strain, γ. These curves are used for performing site-specific response analyses. These shear modulus reduction curves are primarily influenced by the strain amplitude, confining pressure, soil type, and plasticity. The shear modulus reduction curve is typically obtained by using a hyperbolic model. A modified hyperbolic model by Stokoe et al. (1999) has been used by Andrus et al. (2003) to develop shear modulus reduction curves for South Carolina soils. The hyperbolic model by Stokoe et al. (1999) is shown in the following equation.

$$G/G_{max} = \frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}} \qquad \text{Equation 12-19}$$

Where, α is the curvature coefficient, γ, is the shear strain, and γ_r is the reference shear strain. The curvature coefficient, α, and reference shear strain, γ_r, have been estimated by Andrus et al. (2003) to provide the most accurate values for South Carolina Soils. Because it was found that the reference shear strain, γ_r, varied based on effective confining pressure, reference shear strain, γ_r, values are computed using reference shear strain at 1 tsf (100 kPa, 1 atm), γ_{r1}, as shown in the following equation.

$$\gamma_r = \gamma_{r1}\left(\sigma'_m / P_a\right)^k \qquad \text{Equation 12-20}$$

The mean confining pressure, σ'_m, at depth (Z) is computed as shown in Equation 12-21 in units of kPa, where P_a is the reference pressure of 100 kPa, and k is an exponent that varies based on the geologic formation and Plasticity Index, *PI*. Laboratory studies by Stokoe et al. (1995) indicate that the mean confining pressure, σ'_m, values of each layer within a geologic unit should be within ±50 percent of the range of σ'_m for the major geologic unit.

$$\sigma'_m = \sigma'_v \left[\frac{1+2K'_o}{3} \right] \quad \text{Equation 12-21}$$

Where,

σ'_v = vertical effective pressure (kPa)

K'_o = coefficient of effective earth pressure at rest. The K'_o is defined as the ratio of horizontal effective pressure, σ'_h, to vertical effective pressure, σ'_v. The coefficient of effective earth pressure at-rest, K'_o, can be approximated by the coefficient of at-rest pressure, K_o, equations shown in Table 12-14.

Table 12-14, Estimated Coefficient of At-Rest Pressure, K_o

Soil Type	Equation [1]	Equation No.
Normally Consolidated Granular Soils (Jaky, 1944)	$K_0 \approx 1-\sin\phi'$	Equation 12-22
Normally Consolidated Clay Soils (Brooker and Ireland, 1965)	$K_0 \approx 0.95-\sin\phi'$	Equation 12-23
Normally Consolidated Clay Soils (0 < PI ≤ 40) (Brooker and Ireland, 1965)	$K_o \approx 0.40+0.007(PI)$	Equation 12-24
Normally Consolidated Clay Soils (40 < PI < 80) (Brooker and Ireland, 1965)	$K_o \approx 0.6+0.001(PI)$	Equation 12-25
Overconsolidated Clays (Alpan, 1967; Schmertmann, 1975)	$K_o \approx K_{o(N.C.)}\sqrt{OCR}$	Equation 12-26
Overconsolidated Soils (Mayne and Kulhawy, 1982)	$K_o \approx K_{o(N.C.)}OCR^{\sin\phi'}$	Equation 12-27

[1] ϕ'=Drained Friction Angle; *PI*=Plasticity Index; *N.C.*=Normally Consolidated; *OCR* = Overconsolidated Ratio

Values for the reference strain at 1 tsf (100 kPa, 1 atm), γ_{r1}, curvature coefficient, α, and k exponent are provided for South Carolina soils based on Andrus et al. (2003) in Table 12-15.

Table 12-15, Recommended Values γ_{r1}, α, and k for SC Soils (Andrus et al., 2003)

Geologic Age and Location of Deposits [(1)]	Variable	Soil Plasticity Index, *PI* (%)					
		0	15	30	50	100	150
Holocene	γ_{r1} (%)	0.073	0.114	0.156	0.211	0.350	0.488
	α	0.95	0.96	0.97	0.98	1.01	1.04 [(2)]
	k	0.385	0.202	0.106	0.045	0.005	0.001 [(2)]
Pleistocene (Wando)	γ_{r1} (%)	0.018	0.032	0.047	0.067	0.117	0.166
	α	1.00	1.02	1.04	1.06	1.13	1.19
	k	0.454	0.402	0.355	0.301	0.199	0.132
Tertiary Ashley Formation (Cooper Marl)	γ_{r1} (%)	---	---	0.030 [(2)]	0.049	0.096 [(2)]	---
	α	---	---	1.10 [(2)]	1.15	1.28	---
	k	---	---	0.497 [(2)]	0.455	0.362 [(2)]	---
Tertiary (Stiff Upland Soils)	γ_{r1} (%)	---	---	0.023	0.041 [(2)]	---	---
	α	---	---	1.00	1.00 [(2)]	---	---
	k	---	---	0.102	0.045 [(2)]	---	---
Tertiary (All soils at SRS except Stiff Upland Soils)	γ_{r1} (%)	0.038	0.058	0.079	0.106	0.174 [(2)]	---
	α	1.00	1.00	1.00	1.00	1.00 [(2)]	---
	k	0.277	0.240	0.208	0.172	0.106 [(2)]	---
Tertiary (Tobacco Road, Snapp)	γ_{r1} (%)	0.029	0.056	0.082	0.117	0.205 [(1)]	---
	α	1.00	1.00	1.00	1.00	1.00 [(1)]	---
	k	0.220	0.185	0.156	0.124	0.070 [(1)]	---
Tertiary (Soft Upland Soils, Dry Branch, Santee, Warley Hill, Congaree)	γ_{r1} (%)	0.047	0.059	0.071	0.086	0.125 [(1)]	---
	α	1.00	1.00	1.00	1.00	1.00 [(1)]	---
	k	0.313	0.299	0.285	0.268	0.229 [(1)]	---
Residual Soil and Saprolite	γ_{r1} (%)	0.040	0.066	0.093 [(1)]	0.129 [(1)]	---	---
	α	0.72	0.80	0.89	1.01 [(1)]	---	---
	k	0.202	0.141	0.099	0.061 [(2)]	---	---

[(1)] SRS = Savannah River Site
[(2)] Tentative Values – Andrus et al. (2003)

The procedure for computing the G/G_{max} correlation using Equation 12-19 is provided in Table 12-16.

Table 12-16, Procedure for Computing G/G_{max}

Step	Procedure Description
1	Perform a geotechnical subsurface exploration and identify subsurface soil geologic units, approximate age, and formation.
2	Develop soil profiles based on geologic units, soil types, average *PI*, and soil density. Subdivide major geologic units to reflect significant changes in *PI* and soil density. Identify design ground water table based on seasonal fluctuations and artesian pressures.
3	Calculate the average σ'_m and determine the corresponding ±50% range of σ'_m for each major geologic unit using Equation 12-21
4	Calculate σ'_m for each layer within each major geologic unit. If the values for σ'_m of each layer are within a geologic unit's ±50% range of σ'_m (Step 3) then assign the average σ'_m for the major geologic unit (Step 3) to all layers within it. If the σ'_m of each layer within a geologic unit is not within the ±50% range of σ'_m for the major geologic unit, then the geologic unit needs to be "subdivided" and more than one average σ'_m needs to be used, provided the σ'_m remain within the ±50% range of σ'_m for the "subdivided" geologic unit.
5	Select the appropriate values for each layer of reference strain, γ_{r1}, at 1 tsf (1 atm), curvature coefficient, α, and *k* exponent from Table 12-15. These values may be selected by rounding to the nearest *PI* value in the table or by interpolating between listed *PI* values in the table.
6	Compute the reference strain, γ_r, based on Equation 12-20 for each geologic unit (or "subdivided" geologic unit) that has a corresponding average σ'_m.
7	Compute the design shear modulus reduction curves (G/G_{max}) for each layer by substituting reference strain, γ_r, and curvature coefficient, α, for each layer using Equation 12-19. Tabulate values of normalized shear modulus, G/G_{Max} with corresponding shear strain, γ, for use in a site-specific response analysis.

12.3.7 Equivalent Viscous Damping Ratio Curves

Equivalent Viscous Damping Ratio curves are presented in the form of a Soil Damping Ratio, *D* vs. Shear Strain, γ. The Soil Damping Ratio represents the energy dissipated by the soil and is related to the stress-strain hysteresis loops generated during cyclic loading. Energy dissipation or damping is due to friction between soil particles, strain rate effects, and nonlinear behavior of soils. The damping ratio is never zero, even when soils are straining within the linear elastic range of the cyclic loading. The damping ratio, *D*, is constant during the linear elastic range of the cyclic loading and is referred to as the small-strain material damping, D_{min}. The small-strain material damping, D_{min}, can be computed using Stokoe et al., (1995) Equation 12-28.

$$D_{\min} = D_{\min 1}\left(\sigma'_m / P_a\right)^{-0.5k} \qquad \text{Equation 12-28}$$

Where D_{min1} is the small-strain damping at a σ'_m of 1 tsf (1 atm). The mean confining pressure, σ'_m, is computed using Equation 12-21. The *k* exponent is provided for South Carolina soils based on Andrus et al. (2003) in Table 12-15. A relationship for D_{min1} based on soil plasticity index, *PI*, and fitting parameters "*a*" and "*b*" for specific geologic units has been developed by Darendeli (2001) as indicated in Figure 12-27. Values for D_{min1}, small-strain damping @ σ'_m = 1 atm are provided for South Carolina soils based on Andrus et al. (2003) in Table 12-17. The mean confining pressure, σ'_m, at depth (*Z*) is computed as shown in Equation 12-21 in units of kPa.

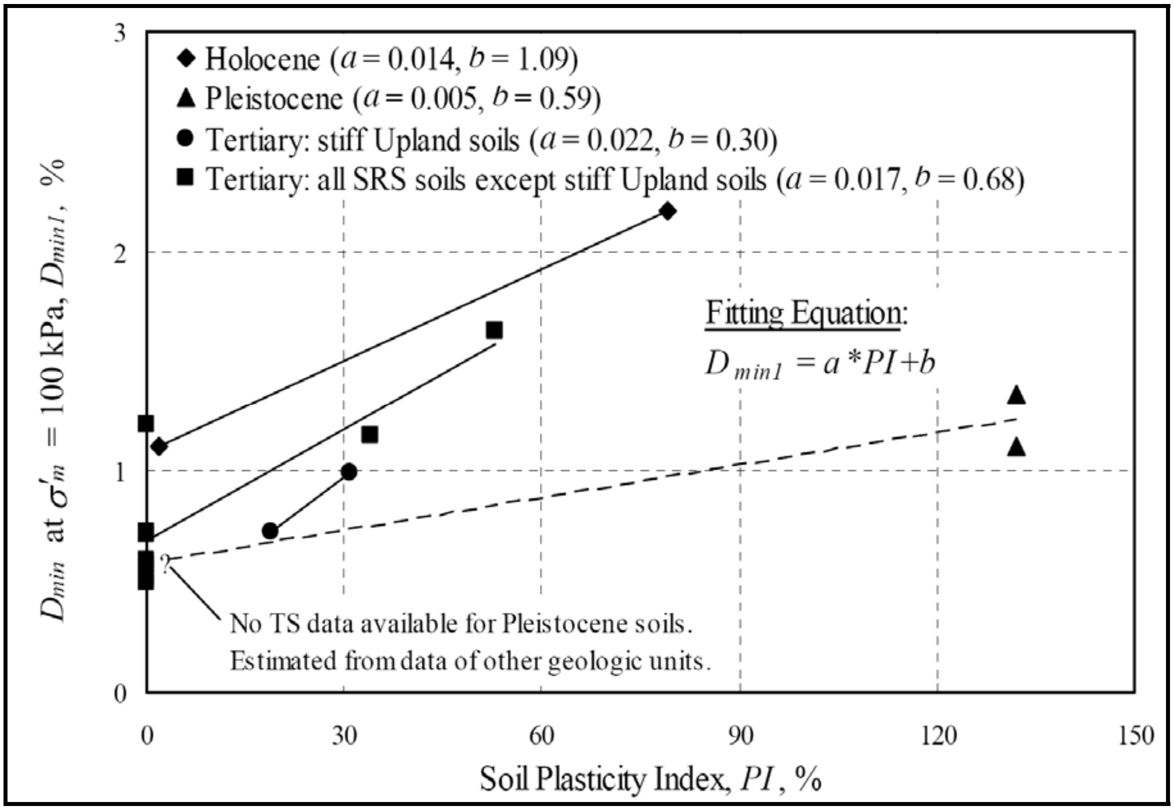


Figure 12-27, D_{min1}, Small-Strain Damping @ σ'_m = 1 atm (Andrus et al., 2003)

Table 12-17, Recommended Value D_{min1} (%) for SC Soils (Andrus et al., 2003)

Geologic Age and Location of Deposits	Soil Plasticity Index, *PI* (%)					
	0	15	30	50	100	150
Holocene	1.09	1.29	1.50	1.78	2.48	3.18 (1)
Pleistocene (Wando)	0.59	0.66	0.73	0.83	1.08	1.32
Tertiary Ashley Formation (Cooper Marl)	---	---	1.14 (1)	1.52 (1)	2.49 (1)	---
Tertiary (Stiff Upland Soils)	---	---	0.98	1.42 (1)	---	---
Tertiary (All soils at SRS except Stiff Upland Soils)	0.68	0.94	1.19	1.53	2.37 (1)	---
Tertiary (Tobacco Road, Snapp)	0.68	0.94	1.19	1.53	2.37 (1)	---
Tertiary (Soft Upland Soils, Dry Branch, Santee, Warley Hill, Congaree)	0.68	0.94	1.19	1.53	2.37 (1)	---
Residual Soil and Saprolite	0.56 (1)	0.85 (1)	1.14 (1)	1.52 (1)	---	---

(1) Tentative Values – Andrus et al. (2003)

Data compiled by the University of Texas at Austin (UTA) for ($D – D_{min}$) vs. (G/G_{max}) is plotted in Figure 12-28.

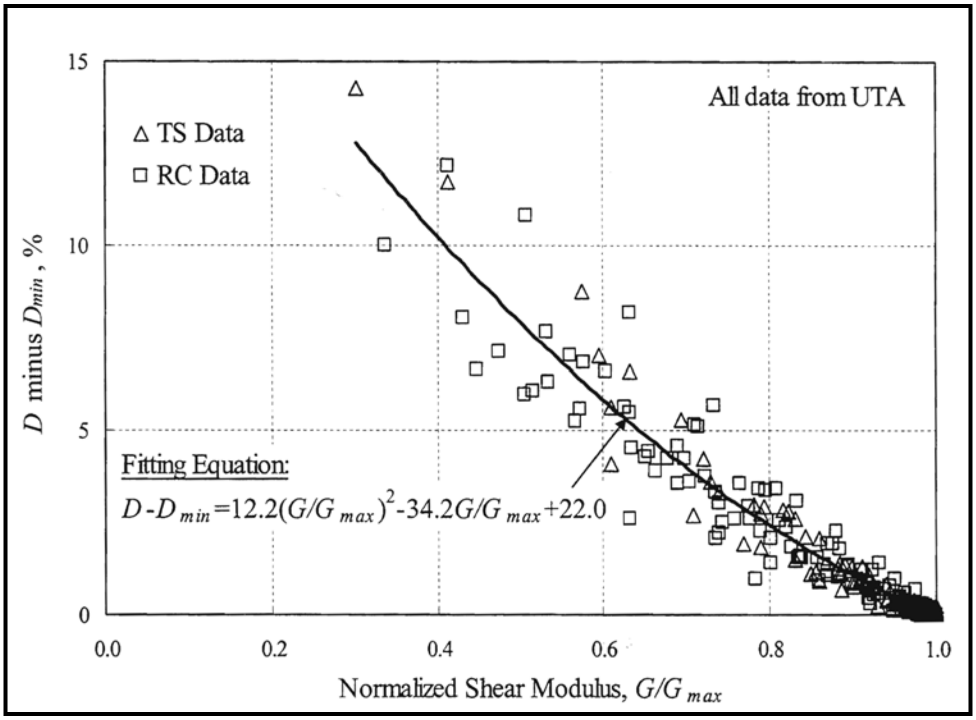


Figure 12-28, ($D – D_{min}$) vs. (G/G_{max}) Relationship (Andrus et al., 2003)

Equation 12-29 represents a best-fit equation (UTA Correlation) of the observed relationship of ($D – D_{min}$) vs. (G/G_{max}) indicated in Figure 12-28.

$$D - D_{min} = 12.2(G/G_{max})^2 - 34.2(G/G_{max}) + 22.0 \qquad \text{Equation 12-29}$$

If we substitute Equation 12-19 into Equation 12-29 and Solve for damping ratio, D, the Equivalent Viscous Damping Ratio curves can be generated using Equation 12-30.

$$D = D_{min} + 12.2\left(\frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}}\right)^2 - 34.2\left(\frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}}\right) + 22.0 \qquad \text{Equation 12-30}$$

Where values of reference strain, γ_r, are computed using Equation 12-20.

The procedures for using Equation 12-30 are provided in Table 12-18.

Table 12-18, Procedure for Computing Damping Ratio

Step	Procedure Description
1	Perform a geotechnical subsurface exploration and identify subsurface soil geologic units, approximate age, and formation.
2	Develop soil profiles based on geologic units, soil types, average *PI*, and soil density. Subdivide major geologic units to reflect significant changes in *PI* and soil density. Identify design ground water table based on seasonal fluctuations and artesian pressures.
3	Calculate the average σ'_m and determine the corresponding ±50% range of σ'_m for each major geologic unit using Equation 12-21.
4	Calculate σ'_m for each layer within each major geologic unit. If the values for σ'_m of each layer are within a geologic unit's ±50% range of σ'_m (Step 3) then assign the average σ'_m for the major geologic unit (Step 3) to all layers within it. If the σ'_m of each layer within a geologic unit is not within the ±50% range of σ'_m for the major geologic unit, then the geologic unit needs to be "subdivided" and more than one average σ'_m needs to be used, provided the σ'_m remain within the ±50% range of σ'_m for the "subdivided" geologic unit.
5	Select appropriate small-strain material Damping @ σ'_m = 1 atm, D_{min1}, from Table 12-17 for each layer within a geologic unit.
6	Compute the small-strain material Damping, D_{min}, for each layer within a geologic unit using Equation 12-28.
7	Select the appropriate values for each layer of reference strain, γ_{r1}, @ σ'_m = 1atm , curvature coefficient, α, and *k* exponent from Table 12-15. These values may be selected by rounding to the nearest *PI* value in the table or by interpolating between listed *PI* values in the table.
8	Compute the reference strain, γ_r, based on Equation 12-20 for each geologic unit that has a corresponding average σ'_m.
9	Compute the design equivalent viscous damping ratio curves (*D*) for each layer by substituting reference strain, γ_r, and curvature coefficient, α, and small-strain material Damping, D_{min}, for each layer using Equation 12-30. Tabulate values of Soil Damping Ratio, *D*, with corresponding shear strain, γ, for use in a site-specific site response analysis.

12.3.8 Alternate Dynamic Property Correlations

12.3.8.1 Soil Stiffness

The SPT and CPT shear wave, V_s, correlations provided in Sections 12.3.2.1 and 12.3.2.2 are based on studies performed by Andrus et al. (2003) for South Carolina soils. If the Andrus et al. (2003) shear wave correlations are not appropriate (i.e. embankment fill) for the soils encountered at a specific project site, the geotechnical engineer can use alternate correlations provided that documentation is provided explaining the use of an alternate correlation and that the correlation is nationally or regionally recognized. Acceptable correlations that can be used are listed in Table 12-19.

Table 12-19, Alternate Correlations of Soil Stiffness (G_{max})

Reference	Correlation Equation	Units	Comments
Seed, et al. (1984)	$G_{max} = 220(K_2)_{max}(\sigma'_m)^{0.5}$ $(K_2)_{max} \approx 20(N_1)_{60}^{1/3}$	kPa	$(K_2)_{max} \approx$ 30 for loose sands and 75 for very dense sands; ≈80-180 for dense well graded gravels; Limited to cohesionless soils
Imai and Tonouchi (1982)	$G_{max} = 15{,}560(N_{60})^{0.68}$	kPa	Limited to cohesionless soils
Hardin (1978)	$G_{max} = \frac{625}{(0.3+0.7e_o^2)}(P_a\sigma'_m)^{0.5} OCR^k$	kPa (1)	Limited to cohesive soils P_a = atmospheric pressure P_a and σ'_m in kPa
Jamiolkowski, et al. (1991)	$G_{max} = \frac{625}{e_o^{1.3}}(P_a\sigma'_m)^{0.5} OCR^k$	kPa (1)	Limited to cohesive soils P_a and σ'_m in kPa
Mayne and Rix (1993)	$G_{max} = 99.5(P_a)^{0.305}\frac{(q_c)^{0.695}}{(e_o)^{1.13}}$	kPa	Limited to cohesive soils P_a and q_c in kPa

(1) The parameter k is related to the plasticity index, PI, as follows:

PI	k	PI	k
0	0.00	60	0.41
20	0.18	80	0.48
40	0.30	>100	0.50

12.3.8.2 Shear Modulus Reduction Curves

The shear modulus reduction curves provided in Section 12.3.6 are based on studies performed by Andrus et al. (2003) for South Carolina soils. If the Andrus et al. (2003) shear modulus reduction curves are not appropriate (i.e. embankment fill) for the soils encountered at a specific project site, the geotechnical engineer may use alternate shear modulus reduction curve correlations provided that documentation is provided explaining the use of the alternate curve and that the alternate curve is nationally or regionally recognized. Acceptable correlations that may be used are listed below:

- Seed and Idriss (1970)
- Vucetic and Dobry (1991)
- Ishibashi and Zhang (1993)
- Idriss (1990)
- Seed et al. (1986)

12.3.8.3 Equivalent Viscous Damping Ratio Curves

The equivalent viscous damping ratio curves provided in Section 12.3.7 are based on studies performed by Andrus et al. (2003) for South Carolina soils. If the Andrus et al. (2003) equivalent viscous damping ratio curves are not appropriate (i.e. embankment fill) for the soils encountered at a project site the geotechnical engineer may use alternate equivalent viscous damping ratio curves provided that documentation is provided explaining the use of the alternate curve and that the alternate curve is nationally or regionally recognized. Acceptable correlations that may be used are listed below:

- Seed et al. (1986)
- Idriss (1990)
- Vucetic and Dobry (1991)

12.4 PROJECT SITE CLASSIFICATION

12.4.1 Site Class Determination

The first step in earthquake engineering is to categorize the project site based on the Site Class. The Site Class of a project site is determined by assigning a Site Class of A, B, C, D, E, or F based on the site stiffness, $\bar{V}_s$, criteria provided in Table 12-22. The site stiffness is a weighted average of the shear wave velocities at the project site. The geotechnical engineer-of-record determines the Site Class based on a careful evaluation of the subsurface investigation and field and laboratory testing results. The project Site Class is determined during the preliminary exploration through the collection of shear wave velocities (Chapters 4 and 5). If the Site Class is required and a preliminary subsurface investigation has not been performed, the geotechnical engineer may use geotechnical information available at the site, past subsurface investigations in the area, and consult geologic maps of the region. After the site-specific geotechnical subsurface investigation has been completed, the preliminary Site Class provided will be re-evaluated and a final Site Class will be provided if necessary.

The site stiffness, $\bar{V}_s$, should be computed in accordance with Section 12.3.4. The total depth (d_T) where shear wave velocities will be analyzed should begin at the anticipated depth-to-motion, Z_{DTM}, and extend to a depth of 100 feet (d_T = 100 ft.) or less if the soil column from the depth-to-motion, Z_{DTM}, to the location where the ground motion is placed using geologically realistic site conditions is located less than 100 feet. When evaluating Site Class C, D, E, or F, the soil column should consist of soils with shear wave velocities less than 2,500 ft/sec. The depth-to-motion is the location where the ground motion transmits the ground shaking energy to the structure being designed. Guidance in selecting the depth-to-motion, Z_{DTM}, is provided in Section 12.4.2.

When there is a high contrast in shear wave velocities in the soil column the computed site stiffness, $\bar{V}_s$, may not be representative of the site response. The geotechnical engineer will need to evaluate the computed site stiffness for high variation in shear wave velocity within the profile that could potentially overestimate the site stiffness and in turn underestimate amplification of the spectral accelerations. The following procedure to evaluate site stiffness, $\bar{V}_s$, variability is to be used cautiously as only a guide. The geotechnical engineer will be responsible for making all site stiffness, $\bar{V}_s$, recommendations, and these recommendations will be submitted to the PCS/GDS for approval. The proposed procedure to evaluate the site stiffness, $\bar{V}_s$, variability is based on the potential variability of shear wave testing having a Coefficient of Variability (COV) of 0.10 to 0.20. The proposed procedure to evaluate site stiffness variability is shown in Table 12-20.

Table 12-20, Site Stiffness Variability Proposed Procedure

Step	Description
1	Compute the Coefficient of Variability (COV) of the shear wave velocity values (COV_{VS}) within the soil profile column. If the COV_{VS} is greater than 0.10 proceed to Step 2. If the $COV_{VS} \leq 0.10$ then compute the site stiffness, $\overline{V}_s$, using the shear wave values (V_s) in accordance with Section 12.3.4 and then determine the Site Class.
2	If $0.10 < COV_{VS} \leq 0.20$ compute the adjusted site stiffness, $\overline{\boldsymbol{V}}_s'$, using Equation 12-31 then proceed to Step 3. $$\overline{\boldsymbol{V}}_s' = \overline{\boldsymbol{V}}_s(1 - 0.20) \qquad \text{Equation 12-31}$$ If $0.20 < COV_{VS} \leq 0.30$ compute the adjusted site stiffness, $\overline{\boldsymbol{V}}_s'$, using Equation 12-32 then proceed to Step 3. $$\overline{\boldsymbol{V}}_s' = \overline{\boldsymbol{V}}_s(1 - \boldsymbol{COV_{VS}}) \qquad \text{Equation 12-32}$$ If $COV_{VS} > 0.30$ the geotechnical engineer shall submit to the PCS/GDS either a recommended (with documentation) site stiffness, $\overline{\boldsymbol{V}}_s$, and Site Class to be used for the project or a request to perform a site-specific response analysis in accordance with Section 12.8.
3	The site stiffness, $\overline{V}_s$,is then computed as follows: $$\overline{\boldsymbol{V}}_s = \overline{\boldsymbol{V}}_s' \qquad \text{Equation 12-33}$$ Use the new site stiffness, $\overline{V}_s$, to determine the Site Class.

When a project site has more than one Site Class due to soil spatial variations along the project alignment or when different structural components (bridge abutment, interior bents, embankments, etc.) require differing depth-to-motion, Z_{DTM}, the designer will need to evaluate the Site Class for each structure component being designed. Guidance in selecting the most appropriate Site Class for the structure being designed can be found in Section 12.4.3.

The steps for determining the project Site Class are described in Table 12-21.

Table 12-21, Site Class Determination Procedure

Step	Description
1	Check for the three criteria of Site Class F shown in Table 12-22 that would require a site-specific response evaluation. If the site meets any of these criteria, classify the project site as Site Class F.
2	Check for the existence of a soft soil layer with a total thickness, H > 10 ft (3 m). A soft soil layer is defined by: $PI > 20$, $w \geq 40\%$, and $\overline{s}_u < 500$ psf (25 kPa). If this criteria is satisfied, the project site is a Site Class E.
3	If a Site Class has not been assigned using Steps 1 and 2 above then compute the site stiffness, $\overline{\boldsymbol{V}}_s$, using the procedures in Section 12.3.4 and Table 12-20.
4	Determine the Site Class based on the site stiffness, $\overline{V}_S$, using Table 12-22.

Table 12-22, Site Class Seismic Category

SITE CLASS	**SOIL PROFILE NAME**	**AVERAGE PROPERTIES IN TOP 100 FT (30 M) Below Z_{DTM}**
		SITE STIFFNESS $\overline{V}_s$
A	Hard Rock	$\overline{V}_s$ > 5,000 ft/sec ($\overline{V}_s$ >1500 m/sec)
B	Rock	2,500 < $\overline{V}_s$ ≤ 5,000 ft/sec (760 < $\overline{V}_s$ ≤ 1500 m/sec)
C	Very Dense Soil and Soft Rock	1,200 < $\overline{V}_s$ 2,500 ft/sec (360 < $\overline{V}_s$ ≤ 760 m/sec)
D	Stiff Soil	600 ≤ $\overline{V}_s$ ≤ 1,200 ft/sec (180 ≤ $\overline{V}_s$ ≤ 360 m/sec)
E	Soft Soil	$\overline{V}_s$ < 600 ft/sec ($\overline{V}_s$ < 180 m/sec)
		Any profile with more than 10 ft (3m) of soft clay defined as: *P*I > 20; w ≥ 40%; and $\bar{\tau} = \bar{\boldsymbol{s}}_{\boldsymbol{u}}$ < 500 psf (25 kPa)
F	Soils Requiring Site Specific Response Evaluation	Any soil profile containing one or more of the following characteristics: 1. Peats and/or highly organic clays (*H*>10 ft [3 m] of peat and/or highly organic clay where *H* = thickness of soil) 2. Very high plasticity clays (*H*>25 ft [8 m] with *PI* > 75) 3. Very thick soft/medium stiff clays (*H*> 120 ft [36 m])

Definitions:

PI = Plasticity Index (AASHTO T89, T90 or ASTM D 4318)
w = Moisture Content (AASHTO T265 or ASTM D 2216)
$\overline{V}_s$ = Average shear wave velocity for the upper 100 ft (30 m) below Z_{DTM}. (ft/sec or m/sec)
$\bar{\tau}$ = Average undrained shear strength ($\bar{\tau} = \bar{\boldsymbol{s}}_{\boldsymbol{u}}$) for cohesive soils in the upper 100 ft (30 m) below Z_{DTM}. (psf or kPa) (AASHTO T208 or T296 or ASTM D2166 or D2850)
Z_{DTM} = Depth-to-motion is the location where the ground motion transmits the ground shaking energy to the structure.

Notes:

(1) The shear wave velocity for rock, Site Class B, shall be either measured on site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Site Class C.

(2) The hard rock, Site Class A, category shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 feet (30m) below Z_{DTM}, surficial shear wave velocity measurements may be extrapolated to assess shear wave velocities.

(3) Site Classes A and B should not be used when there is more than 10 feet (3m) of soil between the rock surface and the depth-to-motion, Z_{DTM}. When rock is encountered within the 100 feet (30m) below the depth-to-motion, Z_{DTM}, and the soil layer is more than 10 feet (3m) use the Site Class pertaining to the soil above the rock.

(4) A Site Class F is not required if a determination is made that the presence of such soils will not result in a significantly higher response of a bridge. Consideration of the effects of depth-to-motion, Z_{DTM}, shall be taken into account when making this determination. Such a determination must be approved by the PCS/GDS.

12.4.2 Depth-To-Motion Effects On Site Class and Site Factors

The Site Class soil profile under evaluation should begin at the anticipated depth-of-motion, Z_{DTM}, for the structure being designed. The depth-to-motion, Z_{DTM}, is the location where the ground motion transmits the ground shaking energy to the structure being designed. When the depth-to-motion is identified, a structure specific Site Class is determined for the soil profile extending 100 feet (30 m) below the depth-to-motion, Z_{DTM}. Typical structures where a Site Class is needed are bridges, roadway embankments, earth retaining systems, and other roadway structures. The depth-to-motion, Z_{DTM}, can affect the Site Class significantly, particularly, for single component soil-structure interaction (SC-SSI) systems such as pile bents, where soft soils with a Site Class E are at the surface with underlying stiff soils with a Site Class D. If the depth-to-motion, Z_{DTM}, were located below the Site Class E soils, a Site Class D would be selected.

When structures are founded on shallow foundations, the depth-to-motion, Z_{DTM}, is typically located at the base of the structure, such as the base of an embankment fill, bottom of a footing, etc. The effects of fill overburden pressures (i.e. embankment fill) over the underlying soils should be included in the Site Class computations.

When structures are founded on deep foundations, the depth-to-motion determination is more complex because of the soil-structure interaction and should be evaluated jointly between the geotechnical engineer and structural engineer. The depth-to-motion, Z_{DTM}, location for deep foundations is at some point below the ground surface depending on the soil-structure components and their horizontal stiffness. Soil-structure interaction can be characterized as either a single component (i.e. soil-pile interaction) or a multi-component (i.e. soil-pile-footing and soil-pile).

A single component soil-structure interaction (SC-SSI) would be a bridge interior bent supported on a spread footing, bridge interior bent supported by a bent cap above the ground and piles or drilled shafts embedded in the ground, or a single bridge column supported by a drilled shaft. The depth-to-motion, Z_{DTM}, for the spread footing case would be the bottom of the footing. The depth-to-motion, Z_{DTM}, for the pile or drilled shaft bridge foundations listed can be estimated as the point-of-fixity typically used by structural engineers in their structural evaluations. The buckling point-of-fixity is typically used for preliminary analyses. The point-of-fixity is the point at which the earth pressures adequately resist a couple created by the moment, resist the lateral shear, or both.

A multi-component soil-structure interaction (MC-SSI) is comprised of various soil-structure system components with each component having different horizontal stiffness as illustrated in Figure 12-29. Figure 12-29 illustrates an embedded pile group footing where the soil-pile-footing system component has a horizontal stiffness and the soil-pile system component has another horizontal stiffness. If the soil-pile-footing horizontal stiffness were considerably greater than the pile group-soil horizontal stiffness as illustrated in Figure 12-29(A), the depth-to-motion, Z_{DTM}, would be at the base of the footing. Conversely, if the soil-pile system component stiffness were greater than the stiffness of the soil-pile-footing system as illustrated in Figure 12-29(B), the depth-to-motion, Z_{DTM}, would be located at some depth below the surface similar to the SC-SSI. The depth-to-motion, Z_{DTM}, for the soil-pile system will need to account for the pile group interaction.

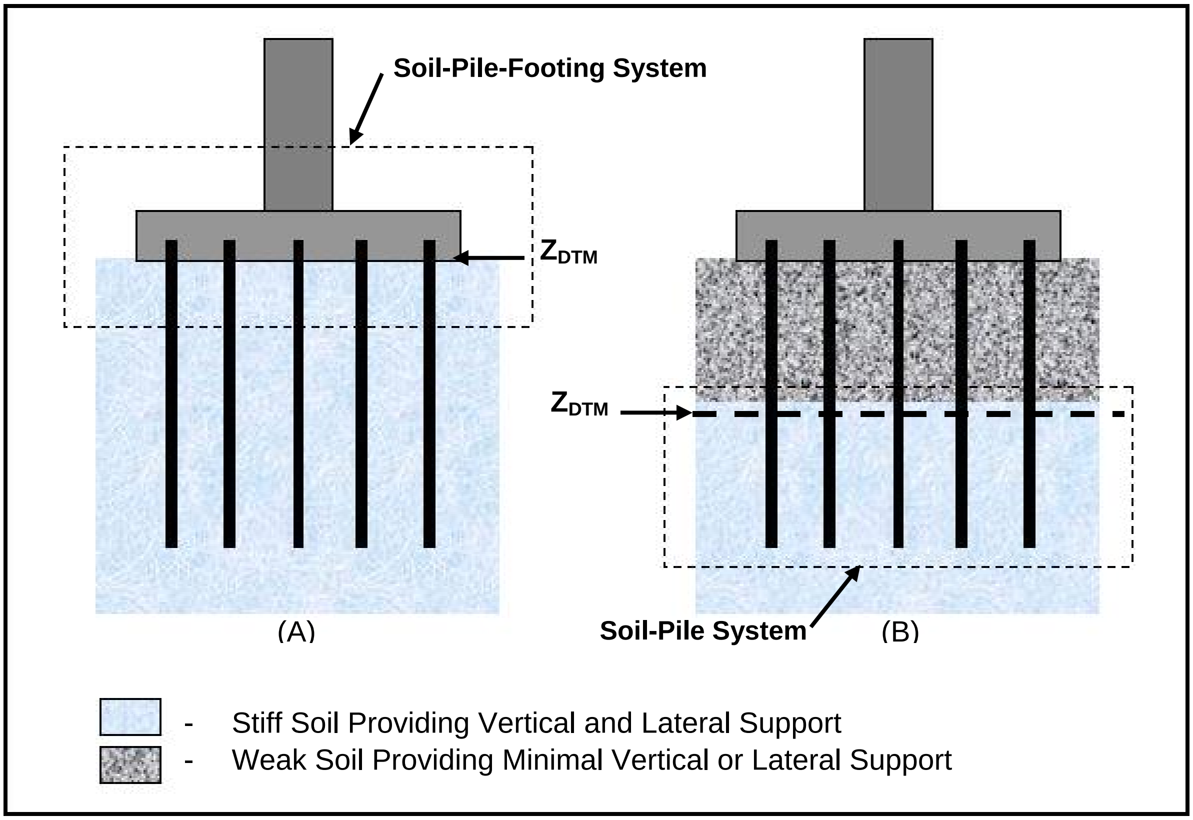


Figure 12-29, Multi-Component Soil-Structure Interaction (MC-SSI)

12.4.3 Site Class Variation Along a Project Site

The procedures for determining Site Class works well when relatively uniform soil conditions are encountered at a project site. As has been seen the depth-to-motion concept discussed in Section 12.4.2 can produce various Site Classes depending on the type of structure or component being analyzed. Using a single Site Class for designing individual structures (bridges, roadway embankments, retaining walls, and miscellaneous roadway structures) can be accomplished by evaluating the primary mechanism by which energy is transferred from the ground to the structure.

If the Site Class varies between the interior bents and abutments of a bridge, the design Site Class of the bridge structure must be evaluated jointly between the geotechnical engineer and the structural engineer. The motion at the bridge abutment for short bridges with relatively few spans will generally be the primary mechanism by which energy is transferred to the bridge superstructure and therefore the Site Class at the bridge abutment would govern. The Site Class for bridges may differ significantly along the bridge alignment due to variability in soil conditions such as one abutment is founded on rock (Site Class B), the other abutment is founded on soft soils (Site Class E), and the interior bents are founded on stiff soils (Site Class D). In this circumstance, the primary mechanism by which energy is transferred to the bridge is more difficult to determine. If only a single site response will be used in the analyses, then an envelope could be developed that captures the predominant periods for the entire spectrum using the various site classes. If the structural analytical method allows the input of several motions at different locations, then several Site Classes should be used.

The geotechnical engineer is responsible for evaluating soil conditions and the extent of site variability (if any) at the bridge location and then determining the Site Class for each individual soil region based on the guidelines provided in this Section. The geotechnical engineer and the structural engineer will then jointly evaluate the appropriate Site Class to be used for the structural design of the bridge.

12.4.4 South Carolina Reference Site Classes

A Site Class was computed for the USGS Shear Wave Velocity Data and SCEMD Seismic Risk and Vulnerability Study based on the shear wave reference profiles in Sections 12.3.3.1 and 12.3.3.2, respectively. The reference Site Class was determined for each shear wave profile using a site stiffness ($\bar{V}_s$) computed in accordance with 12.3.4 for a depth-to-motion at the ground surface (Z_{DTM} = 0).

The site stiffness and corresponding Site Class for the USGS Shear Wave Velocity Data are provided in Table 12-23.

Table 12-23, USGS Site Stiffness and Site Class (Modified Odum et al., 2003)

Site No.	Site Name	Latitude (degrees)	Longitude (degrees)	Surficial Geology [(1)]	Site Stiffness $\bar{V}_S$ (m/s)	Site Stiffness $\bar{V}_S$ (ft/sec)	Site Class [(2, 3)]
1	Lake Murray Spillway	35.052	81.210	Fill, Pz	661	2,168	C
2	Fort Jackson	34.028	90.912	K_u	465	1,525	C
3	Deep Creek School	33.699	79.351	Q?, K_u	246	807	D
4	Black Mingo	33.551	79.933	Q, T_l	477	1,565	C
5	Santee Ls	33.235	80.433	T_l	583	1,912	C
6	The Citadel, Charleston	32.798	79.958	Q, T_u	248	813	D
7	US Hwy. 17, Charleston	32.785	79.955	Fill, Q	182	597	E
8	Isle of Palms	32.795	79.775	Q_h, T_u	179	587	E
9	USNSN	33.106	80.178	Q, T_u	464	1,521	C

[(1)] Definitions: Q – Quaternary; T_u – upper Tertiary; T_l – lower Tertiary; K_u – upper Cretaceous; P_z - Paleozoic

[(2)] Site Classes were evaluated based on Table 12-22 using the shear wave velocities in ft/sec.

[(3)] The depth-to-motion (Z_{DTM} = 0) for the reference Site Class computations was assumed to be the ground surface. Selection of a depth-to-motion below the surface (Z_{DTM} > 0) could significantly affect the Site Class determination.

The site stiffness and corresponding Site Class for the SCEMD Seismic Risk and Vulnerability Study are provided in Table 12-24.

Table 12-24, USGS Site Stiffness and Site Class (Modified URS Corporation, 2003)

Site [1] No.	Site Response Category [1]	Geology	Site Stiffness $\overline{V}_S$ (m/s)	Site Stiffness $\overline{V}_S$ (ft/sec)	Site Class [2, 3]
1, 2, 4 [4]	Piedmont/Blue Ridge, Savannah River, Myrtle Beach [4]	Crystalline	3,400	11,152	A
1	Piedmont/Blue Ridge	Piedmont/Blue Ridge	453	1,486	C
2	Savannah River	Savannah River	355	1,165	D
3	Charleston	Charleston	328	1,077	D
4	Myrtle Beach	Myrtle Beach	239	784	D

(1) Site Response Categories are shown in Figure 12-10.
(2) Site Classes were evaluated based on Table 12-22 using the shear wave velocities in ft/sec.
(3) The depth-to-motion (Z_{DTM} = 0) for the reference Site Class computations was assumed to be the ground surface. Selection of a depth-to-motion below the surface (Z_{DTM} > 0) could significantly affect the Site Class determination.
(4) Various Site Nos. and Site Response Categories are provided for a crystalline geology to account for transition zones between geologies and to allow for any hard-rock basement outcrops located outside of the Piedmont/Blue Ridge Response Category.

12.5 SC EARTHQUAKE HAZARD ANALYSIS

The SC Seismic hazard maps shall be used for all “Typical SCDOT Bridges” as defined by Sections 1.4 and 1.5 of the SCDOT *Seismic Design Specifications for Highway Bridges*. For non-typical bridges, the PCS/GDS will specify and/or approve appropriate geotechnical earthquake engineering provisions on a project specific basis. The SC Seismic Hazard maps are described in Section 11.9.2, Probabilistic Earthquake Hazard Maps. The seismic hazard information generated from these maps includes the PGA and PSA for 0.5Hz, 1.0Hz, 2.0Hz, 3.3Hz, 5Hz, 6.7Hz, and 13Hz frequencies for the FEE and SEE design earthquakes at hard rock basement outcrop or at geologically realistic site condition.

12.6 ACCELERATION RESPONSE SPECTRUM

The acceleration response spectrum of a specific earthquake motion is a plot of the maximum spectral acceleration, S_a, response of a series of linear single degree-of-freedom systems with the same damping and mass, but variable stiffness. The South Carolina Seismic Hazard maps generate a probabilistic Uniform Seismic Hazard of PGA and PSA at either a hard-rock basement outcrop or at a geologically realistic site conditions (i.e. B-C Boundary in the Coastal Plain). The response spectrum at these locations needs to be adjusted for the local site effects. The local site effects are influenced by the soil stiffness (resonant frequency) of the soil column above the location where ground motion was generated. The soil column extends to the location where the ground motion transmits the ground shaking energy to the structure being designed, also referred to as the depth-to-motion, Z_{DTM}, in Section 12.4.2.

The maximum local site amplification occurs when the predominant or maximum period, T_{max}, of the rock outcrop ground motion, the soil deposit's natural period, T_N, and the fundamental period of the structure, T_s, are all in phase. The relationship between rock outcrop and soil surface motions is complex and depends on numerous factors including the fundamental period of the soil profile, strain dependency of soil stiffness and damping, and the characteristics of the rock outcrop motion (Seed and Idriss, 1982).

The effects of local soil site conditions such as rock outcrop, stiff site conditions, soft to medium clay and sand, and deep cohesionless soils on the response spectra shapes (5% damped) are shown in Figure 12-30 (Seed et, al., 1976). Normalized spectral shapes were computed by dividing the spectral acceleration by the peak ground acceleration (PGA) at the surface. These spectral shapes were computed from motion records made on rock and soil sites at close distances to earthquakes ($6 \leq M_w \leq 7$). These normalized spectral curves show that spectral response amplification is significantly greater at longer periods ($\approx$ 1 second) with soil site conditions that have decreasing soil site stiffness. The observed variations in spectral response as a function of subsurface site conditions underscore the importance of properly evaluating the project Site Class in accordance with Section 12.4.

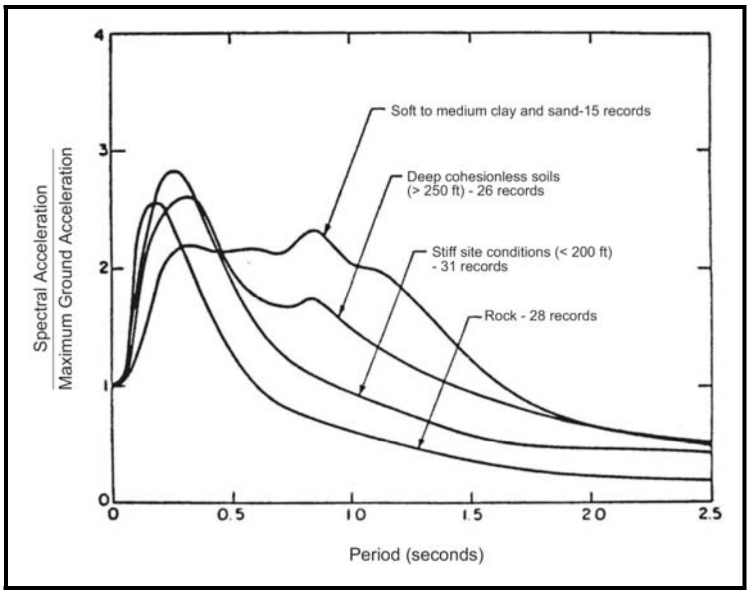


Figure 12-30, Soil Site Effects on Average Normalized Response Spectra (Seed et al., 1976)

Amplification of peak accelerations from the Uniform Hazard occurs when the resonant frequency, f_o, of the soil deposit is close to the predominant frequency or maximum period, T_{max}, of ground motions at either the B-C Boundary or hard-rock basement outcrop.

The natural period, T_N, of the site can be estimated by the following equation.

$$T_N = \frac{1}{f_o} = \frac{4H}{V^*_{s,H}} \qquad \text{Equation 12-34}$$

Where,

- f_o = resonant frequency of the soil deposit thickness (H). Units Hz
- $V^*_{S,H}$ = equivalent uniform soil profile stiffness of thickness (H). Units ft/sec (Section 12.3.4)
- H = thickness of soil deposit above B-C Boundary or hard-rock basement outcrop depending on the level where ground motion input has been developed. Units feet

As can be seen by Equation 12-34, the natural period of the site (T_N) is influenced by the site equivalent uniform soil profile stiffness and the thickness of the soil deposit (H). A general trend is observed in Figure 12-31 that the natural period of a site (T_N) increases as the site stiffness decreases while keeping the soil deposit thickness the same. In addition, as the thickness of the soil profile increases (keeping the site stiffness the same), the natural period of the site increases again. Consequently, a combination of lower site stiffness and increased soil deposit thickness will work together to increase the natural period of the site. At the same time, a reduction in the natural period of the site is observed primarily when the equivalent uniform soil profile stiffness ($V^*_{S,H}$) increases as the depth of the soil profile decreases.

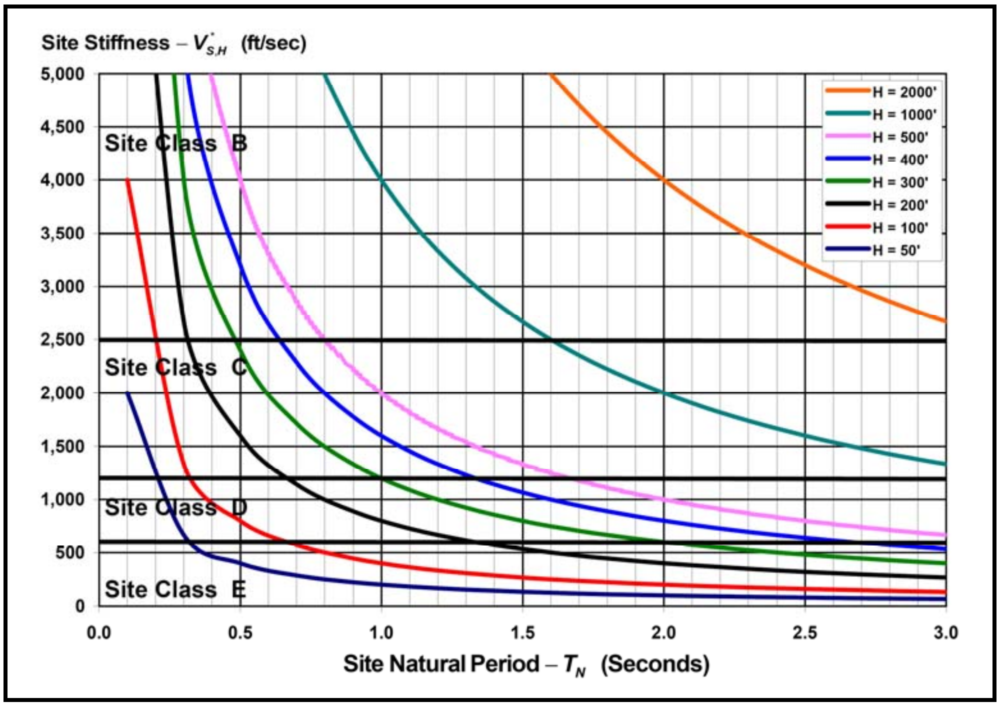


Figure 12-31, Site Natural Period (T_N)

A recent study by Green (2001) reveals that the maximum period, T_{max}, of the bedrock motion in the Central and Eastern United States (CEUS) varies $0.05 < T_{max} < 0.10$ sec. as compared to the Western United States (WUS) which varies $0.15 < T_{max} < 0.25$ sec. This would indicate that South Carolina sites with low natural periods, T_N, in the range of 0.075 seconds would be subject to greater amplification.

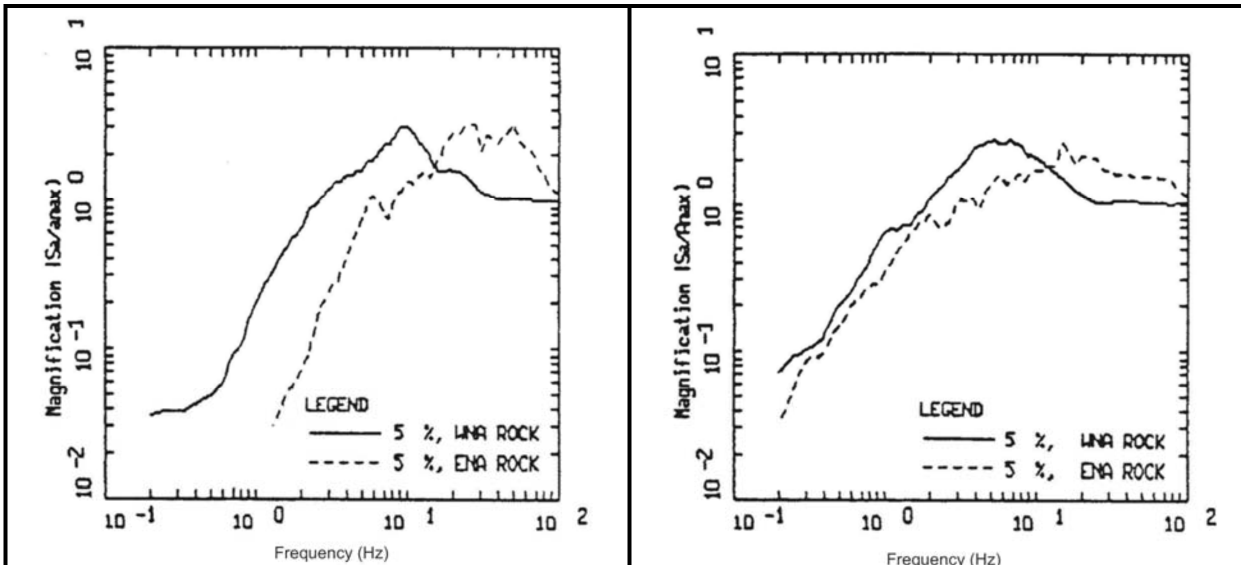
It is equally important to know the fundamental period of the structure (i.e. bridge, earth retaining structure, dam, etc.) being designed since structures with periods similar to the period of the ground motion reaching the structure will tend to exert higher seismic loads (demand) and potentially cause significant damage to the structure.

The local site effects are taken into account by performing a site response analysis using the SC Seismic Hazard Maps (Section 12.7) or by performing a site-specific response analysis (Section 12.8).

The following subsections 12.6.1, 12.6.2, and 12.6.3 describe special site conditions that may influence the site response that typically cannot be addressed by simplified response methods that use the SC Seismic Hazard Maps (Section 12.7).

12.6.1 Effects of Rock Stiffness WNA vs. ENA

The effects of rock stiffness (shear wave velocity) and damping on normalized response spectra shapes (5% damped) on rock sites are shown in Figure 12-32 (Silva and Darragh, 1995). Normalized spectral shapes were computed by dividing the spectral acceleration by the peak ground acceleration (PGA) at the surface. Normalized response spectra were computed for Western North America (WNA), representative of soft rock encountered in California and for Eastern North America (ENA), representative of hard rock encountered in the Eastern United States. The normalized response spectra were computed from motion records made on rock sites at close distances to earthquakes (M_w = 4.0 and 6.4). These normalized spectral curves show that ENA spectral response amplification is greater at longer periods when compared to WNA spectral response. This effect of higher amplification at longer periods is more evident for smaller earthquakes because of higher corner frequencies for smaller magnitude earthquakes (Boore, 1983; Silva and Green, 1989; Silva and Darragh, 1995).



Earthquake M_w ~ 4.5	Earthquake M_w ~ 6.5

Figure 12-32, WNA / ENA Rock Effects on Normalized Response Spectra (Silva and Darragh, 1995)

12.6.2 Effects of Weathered Rock Zones Near the Ground Surface

Some caution should be exercised when evaluating the site response of sites where weathered rock zones are near the surface such as in the Blue Ridge/Piedmont Units and in transition areas between the Piedmont Unit and the Coastal Plain Unit. Transition areas between physiographic units can be found in the Columbia, SC metropolitan area. The Columbia, SC area generally consists of 10 to 30 feet of surficial soils ($200 \leq V_s \leq 500$ ft/sec), underlain by 30 to 90 feet of a weathered rock zone ($2,500 < V_s < 8,000$ ft/sec), followed by a hard-rock basement outcrop (V_s >11,000 ft/sec). A recent site-specific response study (Chapman, 2008) of the Columbia, SC area compared spectral accelerations modeled at a B-C boundary (weathered rock) outcropping conditions and hard-rock outcropping conditions with a weathered rock zone modeled by a shear wave velocity gradient from 2,500 to 8,000 ft/sec on 1.5 ft. increments. This study found that the spectral accelerations for the two models were similar for frequencies up to 10 Hz. (periods > 0.10 seconds). The spectral accelerations increased for frequency greater than 10 Hz. (periods < 0.10 seconds) for the model with hard-rock outcropping conditions and a velocity graded weathered rock zone. The magnitude of the increase in spectral acceleration was dependent on the thickness of the graded weathered rock zone.

Based on this study (Chapman, 2008) the following preliminary guidelines are provided:

1. **Coastal Plain Unit with sedimentary surface soils:** When ground motions are generated using a geologically realistic site condition using Senario_PC (2006) the thickness of the firm Coastal Plain sediment and/or weathered rock zone will be modeled approximately by the transfer function that places the ground motion at the B-C boundary (V_s = 2,500 ft/sec) and therefore the amplification observed from weathered rock thickness greater than 30 feet will not be as significant.

2. **Blue Ridge/Piemont Unit with Weathered Rock Zone:** The Three-Point site response method can only be used if the weathered rock thickness ($2,500 \leq Vs \leq 8,000$ ft/sec) is less than 30 feet thick. When performing site-specific response analyses in the Blue Ridge/Piedmont units with weathered rock zone ($2,500 \leq Vs \leq 8,000$ ft/sec) thickness greater than 30 feet, this zone must be modeled by a shear wave velocity gradient. If the thickness (d_{WR}) of the weathered rock zone is unknown, a sensitivity analysis of the thickness will be required to determine the amplification effects on the spectral accelerations and PGA.

12.6.3 Effects of Soil Softening and Liquefaction on Spectral Acceleration

Youd and Carter (2005) have studied the effects of soil softening and liquefaction on spectral accelerations of five instrumented sites. Three of the sites were in the United States (California) and the other two in Japan. Youd and Carter (2005) made the following observations:

1. Soil softening due to increased pore water pressure generally reduces short period spectral accelerations (T < 1.0 sec) as compared to those spectral accelerations that would have occurred without soil softening.

2. Soil softening may have little influence on short period spectral accelerations (T < 1.0 sec) when soil softening occurs late in the strong motion sequence.

3. Soil softening usually amplifies or enhances long period spectral accelerations (T > 1.0 sec) due to lengthening of the natural period of the site as it softens (See Figure 12-31). When liquefaction-induced ground oscillations continue after earthquake shaking, there may be considerable enhancement of the long-period (T > 1.0 sec) spectral accelerations.

When a site-specific response analysis is not performed and the simplified response methods that use the SC Seismic Hazard Maps (Section 12.7) are used, the effects of soil softening and liquefaction on the design spectral response generated will have the following implications to the structures being designed.

1. For structures with short-fundamental periods (T < 1.0 sec), the design spectral accelerations will conservatively envelope the actual spectral acceleration for sites where soil softening or liquefaction occurs early in the strong motion sequence.

2. For structures with long-fundamental periods (T > 1.0 sec), the design spectral accelerations may be <u>unconservative</u> due to the lengthening of the natural period of the site. For these types of structures with long-fundamental periods (T > 1.0 sec), a site-specific response analysis should be considered.

12.6.4 Horizontal Ground Motion Response Spectra

The SCDOT *Seismic Bridge Design Specifications* requires safety and functional evaluations for bridges based on the bridge Operational Classification, OC. All bridges (OC = I, II, or III) require a structural response evaluation using the Safety Evaluation Earthquake (SEE). Bridges with an OC = I or II also require a structural evaluation using the Functional Evaluation Earthquake (FEE) only if the project site has the potential for liquefaction or slope instability at bridge abutments and no geotechnical mitigation is performed.

The horizontal acceleration design response spectrum (ADRS) curves can be determined by either the Three-Point method (Section 12.7) or the Site-Specific response analysis (Section 12.8) using the selection criteria in Table 12-25.

Table 12-25, Site Response Selection Criteria

Physiographic Unit [1]	Site Response Method	Site Class	Weathered Rock Thickness d_{WR} [2] (feet)	Site Condition	Outcrop Description	Comments
Coastal Plain Unit	Three-Point	A, B, C, D, E	d_{WR} Any	Geologically Realistic	V_S = 2,500 ft/sec B-C Boundary	Site Class should be based on soil column consisting of sediment soils [2] (V_s < 2,500 ft/sec). Document Site Stiffness selection (Table 12-22).
	Site-Specific Response	F or Required by SCDOT	d_{WR} Any	Geologically Realistic	V_S = 2,500 ft/sec B-C Boundary	Soil column model must extend to hypothetical firm Coastal Plain outcrop equivalent of B-C boundary (V_s >2,500 ft/sect). Document soil column properties and soil stratification sensitivity.
				Hard-Rock Basement Outcrop	V_S = 11,500 ft/sec Top of "A" Boundary	Use of ground motions generated at the hard-rock basement outcrop will require written permission by the PCS/GDS. The soil column model must extend to hard-rock basement outcrop (V_s>11,500 ft/sec). The soil column development must be documented thoroughly and extensive soil stratification sensitivity analyses must be performed, particularly below the B-C boundary.
Outside Coastal Plain Unit	Three-Point	A, B, C, D, E	0 < d_{WR} < 30	Geologically Realistic	V_S = 8,200 ft/sec Top of "A" Boundary	Site Class should be based on soil column consisting of sediment soils [2] (V_s < 2,500 ft/sec). Document Site Stiffness selection (Table 12-22). Site Class A must not be used. Select Site Class B only if depth-to-motion (Z_{DTM}) is at top of weathered rock zone (V_s > 2,500 ft/sec)
			d_{WR} = 0	Hard-Rock Basement Outcrop	V_S = 11,500 ft/sec Top of "A" Boundary	Select Site Class A only if depth-to-motion (Z_{DTM}) is at top of hard-rock zone (V_s > 11,000 ft/sec). Note that hard-rock must be verified by shear wave velocity measurements of hard-rock (V_s >11,000 ft/sec).
	Site Specific Response	F or Required by SCDOT	d_{WR} ≤ 30	Geologically Realistic	V_S = 8,200 ft/sec Top of "A" Boundary	Soil column model must extend to hypothetical firm Coastal Plain outcrop equivalent of a hypothetical outcrop of Piedmont weathered rock (Vs >8,000 ft/sect). Document soil column development and soil stratification sensitivity.
			d_{WR} > 30	Hard-Rock Basement Outcrop	V_S = 11,500 ft/sec Top of "A" Boundary	Soil column model must extend to hard-rock basement outcrop (Vs>11,000 ft/sec). Document soil column development and soil stratification sensitivity. The weathered rock zone (2,500 ≤ V_s ≤ 11,500 ft/sec) must be modeled by a shear wave velocity gradient. If thickness (d_{WR} > 30 ft.) of the weathered rock zone is unknown, a sensitivity analysis of the thickness will be required Select Site Class B only if depth-to-motion (Z_{DTM}) is at top of weathered rock zone (V_s > 2,500 ft/sec)

(1) If Senario_PC (2006) indicates a zero sediment thickness (d_S = 0) the site is assumed to be outside of the Coastal Plain (Blue Ridge/Piedmont). If the sediment thickness is greater than zero (d_S > 0) the site is assumed to be in the Coastal Plain.

(2) Weathered rock zone with shear wave velocities 2,500 – 8,000 ft/sec.

Horizontal acceleration design response spectrum (ADRS) curves described in Sections 12.7 and 12.8 are generated for the design earthquakes (SEE and/or FEE) as needed for the structural engineer to perform a structural evaluation. The horizontal ADRS curves are supplied to the structural engineer in the form of a curve and tabulated values of spectral accelerations, S_a, in units of gravity (g) and corresponding time period, T, in units of seconds.

12.6.5 <u>Vertical Ground Motion Response Spectra</u>

Recent studies shown in Figure 12-33 reveal that the ratio of vertical to horizontal ground motion response spectra can vary substantially from the nominal two-thirds (2/3) ratio commonly used. Studies show that the two-thirds ratio of vertical to horizontal ground motion response spectra may be conservative for periods of vibration longer than 0.2 seconds. For periods of vibration shorter than 0.2 seconds the ratio of vertical to horizontal ground motion response spectra may exceed the two-thirds value and may be on the order of 1 to 1.5 times the horizontal for earthquakes with close source-to-site distances and periods of vibration of less than 0.1 seconds. Although the studies shown in Figure 12-33 are from ground motion data from the western United States (WUS), Chiou et al. (2002) indicates that the ratios for the Central and Eastern United States (CEUS) are not greatly different from the ratios in the WUS.

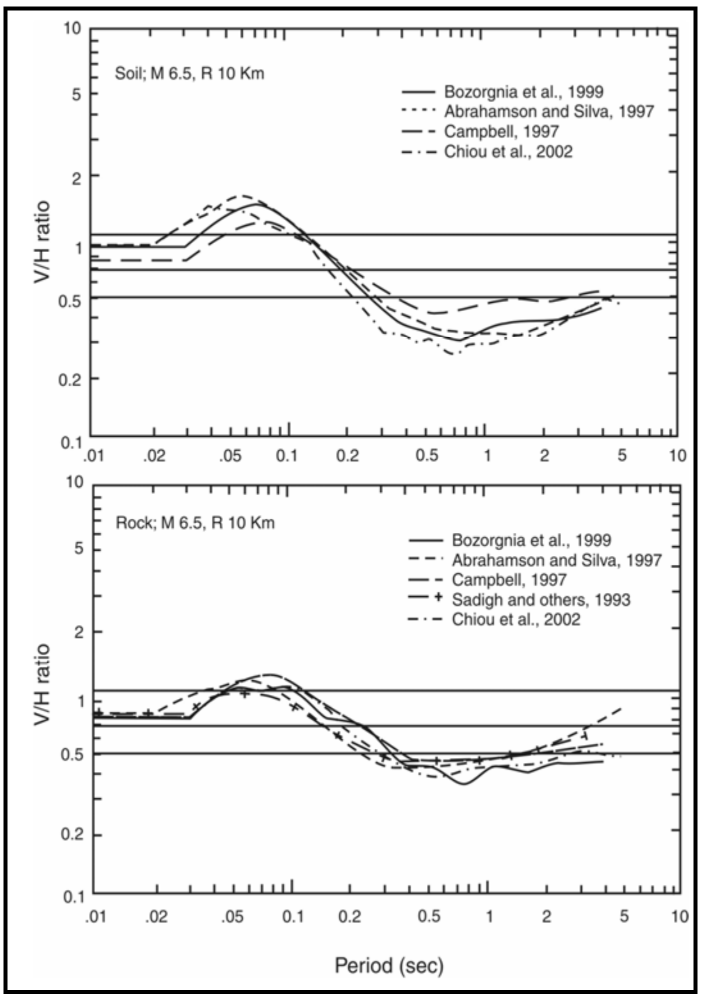


Figure 12-33, Vertical/Horizontal Spectral Ratios vs. Period (Buckle et al, 2005)

Because there are currently no accepted procedures for constructing the vertical response spectra or having an appropriate relationship with the horizontal response spectra constructed using the SC Seismic Hazard maps, Section 12.7, the two-thirds ratio of vertical-to-horizontal response spectra shall be used for bridges with natural periods of vibration of 0.2 seconds or longer. When the bridge's natural period of vibration is less than 0.2 seconds, a site-specific vertical response spectra using the results of recent studies such as those shown in Figure 12-33 should be used to develop the vertical ground motion response spectra.

12.7 SC SEISMIC HAZARD MAPS SITE RESPONSE ANALYSIS

12.7.1 ADRS Curves for FEE and SEE

As described in Section 12.6.2 there are two design earthquakes that are used for evaluation of SCDOT structures, the Functional Evaluation Earthquake (FEE) and the Safety Evaluation Earthquake (SEE). The PGA and spectral response accelerations used in Section 12.7.2 will depend on which design earthquake is being analyzed.

The horizontal ADRS curves generated using the SC Seismic Hazard maps will be based on a 5% viscous damping ratio because the pseudo spectral accelerations (PSA) obtained from the SC Seismic Hazard maps have been generated for 5% damping.

12.7.2 Local Site Effects on PGA

The peak ground acceleration at the existing ground surface is determined by evaluating the local site effects on the mapped peak ground acceleration at the B-C boundary, PGA_{B-C}. The PGA_{B-C} shall be obtained for the appropriate design earthquake (FEE or SEE) being analyzed. The PGA_{B-C} value shall be generated from the SC Seismic Hazard maps as indicated in Sections 12.5 and 11.9.2 at the B-C boundary. The PGA shall be determined by adjusting the PGA_{B-C} based on Site Class using the following equation.

$$PGA = F_{PGA} \cdot PGA_{B-C} \qquad \text{Equation 12-35}$$

Where:

PGA = peak ground acceleration at the existing ground surface (period, T = 0.0 sec.) adjusted for local site conditions

PGA_{B-C} = mapped peak ground acceleration at the B-C boundary (period, T = 0.0 sec.)

F_{PGA} = site coefficient defined in Table 12-26, based on the Site Class and the mapped peak ground acceleration, PGA_{B-C}.

Table 12-26, F_{PGA} Site Factor for Peak Ground Acceleration (PGA)

Site Class	Mapped Peak Ground Acceleration (Period, T = 0 sec.), PGA_{B-C} [1]				
	$PGA_{B-C} \leq 0.10$	$PGA_{B-C} = 0.20$	$PGA_{B-C} = 0.30$	$PGA_{B-C} = 0.40$	$PGA_{B-C} \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[2]	N/A	N/A	N/A	N/A	N/A

Notes:
(1) Use linear interpolation for intermediate values of *PGA*.
(2) Site-specific response analysis shall be performed.

12.7.3 Local Site Effects on Spectral Response Accelerations

The design spectral response accelerations for short period (T = 0.20 second), S_{DS}, and the long period (T = 1.0 second), S_{D1}, at the ground surface are determined by evaluating the local site effects on the horizontal spectral response accelerations for short period (0.20 second), S_S, and long period (1.0 second), S_1, at the B-C boundary. The horizontal spectral accelerations S_s and S_1 values shall be obtained for the appropriate design earthquake (FEE or SEE) being analyzed. The S_s and S_1 values are generated from the SC Seismic Hazard maps as shown in Sections 12.5 and 11.9.2 at the B-C boundary (geologically realistic). Design spectral response accelerations S_{DS} and S_{S1} shall be determined using Equation 12-36 and Equation 12-40, respectively.

$$S_{DS} = F_a S_S \quad \text{Equation 12-36}$$

$$S_{D1} = F_v S_1 \quad \text{Equation 12-37}$$

Where:

- S_{DS} = design short-period (0.2-second) spectral response acceleration parameter
- S_{D1} = design long-period (1.0 second) spectral response acceleration parameter
- F_a = site coefficient defined in Table 12-27, based on the Site Class and the mapped spectral acceleration for the short-period, S_S.
- F_v = site coefficient defined in Table 12-28, based on the Site Class and the mapped spectral acceleration for the long-period, S_1.
- S_S = the mapped spectral acceleration for the short-period (0.2-second) as determined in Sections 12.5 and 11.8.2 at the B-C boundary
- S_1 = the mapped spectral acceleration for the one second period as determined in Sections 12.5 and 11.8.2 at the B-C boundary

Table 12-27, F_a Site Factor for Short-Period (0.2 sec = 5 Hz)

Site Class	Mapped Spectral Acceleration at Short-Periods (0.2 sec), S_s [(1)]				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F [(2)]	N/A	N/A	N/A	N/A	N/A

Notes:
(1) Use linear interpolation for intermediate values of S_S.
(2) Site-specific response analysis shall be performed.

Table 12-28, F_v Site Factor for Long-Period (1.0 sec = 1 Hz)

Site Class	Mapped Spectral Acceleration at Long-Period (1.0 sec), S_1 [(1)]				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F [(2)]	N/A	N/A	N/A	N/A	N/A

Notes:
(1) Use linear interpolation for intermediate values of S_1.
(2) Site-specific response analysis shall be performed.

12.7.4 Three-Point Acceleration Design Response Spectrum

The Three-Point method of constructing the horizontal ADRS curve is typically used for structures having natural periods of vibration between 0.2 second and 3.0 second. The Three-Point method has been shown by Power et al. (1997, 1998) to be unconservative in the CEUS for periods between 1.0 second and 3.0 seconds, and a Site Class B (Rock). When the fundamental period of the structure is less than 0.2 seconds or greater than 3.0 seconds, a site-specific response analysis as described in Section 12.8 may be required. The Multi-Point methods shall be used to evaluate the reasonableness of the Three-Point ADRS Curve as discussed in Section 12.7.5. Guidelines for constructing the Three-Point ADRS Curve are illustrated in Figure 12-34 and step-by-step instructions are provided in Table 12-29.

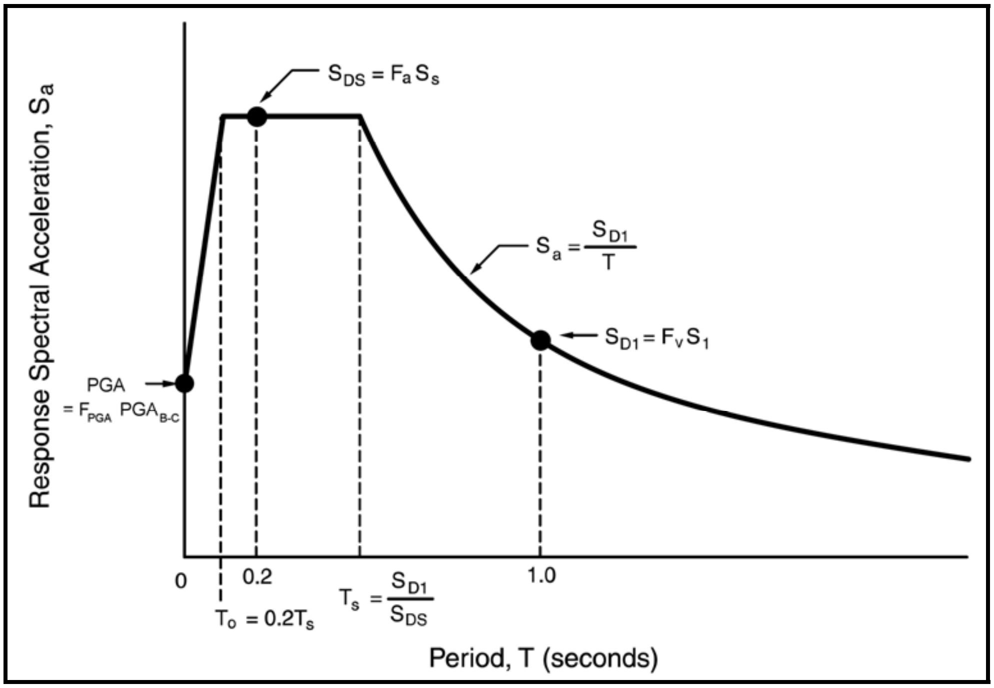


Figure 12-34, Three-Point ADRS Curve

Table 12-29, Three-Point ADRS Construction Procedures

Step	Procedure Description
1	The design short-period acceleration, S_{DS}, at period, T = 0.2 second is computed by using Equation 12-36 from Section 12.7.3. The design long-period acceleration, S_{D1}, at period, T = 1.0 second is computed by using Equation 12-37 from Section 12.7.3. $$S_{DS} = F_a S_S$$ Equation 12-36 $$S_{D1} = F_v S_1$$ Equation 12-37 Where values of F_a, F_v, S_s, and S_1 are obtained as indicated in Section 12.7.3.
2	Period markers T_o and T_s used in constructing the ADRS curves are defined by the following equations. $$T_o = 0.20T_s$$ Equation 12-38 $$T_s = \frac{S_{D1}}{S_{DS}}$$ Equation 12-39 Where S_{DS} and S_{D1} are obtained in Step 1.
3	The PGA at the existing ground surface at period, T=0.0 second is computed by using Equation 12-35 from Section 12.7.2. $$PGA = F_{PGA} \cdot PGA_{B-C}$$ Equation 12-35 Where F_{PGA} and PGA_{B-C} are obtained as indicated in Section 12.7.2.
4	The design spectral response acceleration S_a for periods, $T \leq T_o$, is computed by the following equation. $$S_a = PGA + \left[\left(S_{DS} - PGA \right) \left(\frac{T}{T_o} \right) \right]$$ Equation 12-40 Where, S_{DS} is obtained in Step 1, T_o is obtained in Step 2, and PGA is obtained in Step 3.
6	The design spectral response acceleration, S_a, for periods, $T_o \leq T \leq T_s$, is taken equal to S_{DS}, as obtained in Step 1.
7	The design spectral response acceleration, S_a, for periods, $T_s > T \leq 3.0$ seconds, is computed by the following equation. Equation 12-41 $$S_a = \frac{S_{D1}}{T}$$ Where, S_{D1} is obtained in Step 1.

12.7.5 Multi-Point Acceleration Design Response Spectrum

The Multi-Point method of constructing an ADRS curve shall be used to check the reasonableness of the Three-Point ADRS curve. This is accomplished by first constructing the Three-Point ADRS curve and then overlaying on the same graph the Multi-Point ADRS values as shown in Figure 12-35. The designer should be aware that Power and Chiou (2000) have found that the Multi-Point method may give ambiguous results for structures on sites other than rock (Site Class B). This is due to the Multi-Point method using the short period (0.2 seconds) site factor F_a for all the PSA values with periods less than or equal to 0.2 seconds and using long-period (1.0 seconds) site factor, F_v, for all periods greater than or equal to 1.0 seconds to compute the acceleration response spectrum. The Multi-Point method has been found to be appropriate for structures located on rock (Site Class B) because the site factors (F_a and F_v) for Site Class B are all unity, therefore no amplification or damping. Since the Multi-Point method is only used to check the reasonableness of the Three-Point ADRS curve, the construction of the Multi-Point ADRS Curve for Site Classes other than "B" should be adequate. Guidelines for constructing the Multi-Point ADRS curve are provided in Table 12-30.

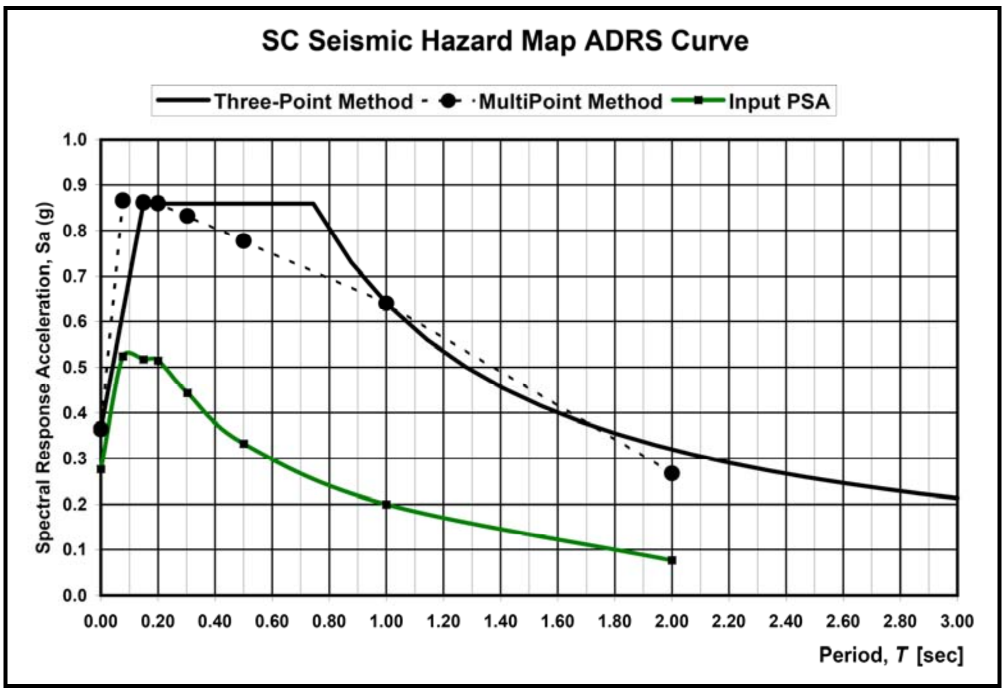


Figure 12-35, Three-Point/Multi-Point ADRS (Site Class=C)

Table 12-30, Multi-Point ADRS Construction Procedure

Step	Procedure Description
1	The FEE or SEE mapped pseudo spectral accelerations (PSA) for periods, T = 2.0 sec (0.5Hz), 1.0 sec (1.0Hz), 0.20 sec (5Hz), 0.15 sec (6.7Hz), 0.08 sec (13Hz) and PGA (PGA_{B-C}) are obtained from the SC Seismic Hazard map as indicated in Sections 12.5 and 11.9.
2	The PGA at the existing ground surface (Period, T=0) is computed by using Equation 12-35 from Section 12.7.2. $$PGA = F_{PGA} \cdot PGA_{B-C} \qquad \text{Equation 12-35}$$ Where F_{PGA} and PGA_{B-C} are obtained as indicated in Section 12.7.2 and Step 1, respectively.
3	The design spectral response acceleration, S_a, for periods, $0.00 < T \leq 0.20$ second is computed using the following equation. $$S_a(T) = F_a S_{\leq 0.20} \qquad \text{Equation 12-42}$$ Where $S_{\leq 0.20}$ includes PSA for periods, T = 0.08 sec (13Hz), 0.15 sec (6.7Hz), and 0.20 sec (5Hz) from Step 1. The site factor F_a is obtained as indicated in Section 12.7.3
4	The design spectral response acceleration, S_a, for periods, $1.0 \leq T \leq 3.0$ second is computed using the following equation. $$S_a = F_v S_{\geq 1.0} \qquad \text{Equation 12-43}$$ Where $S_{\geq 1.0}$ includes PSA for 1.0 sec (1.0Hz) and 2.0 sec (0.5Hz). The site factor F_v is obtained as indicated in Section 12.7.3.
5	The spectral accelerations, S_a, for periods, $0.20 < T < 1.0$ sec should be linearly interpolated between $S_{0.20}$ at T= 0.20 seconds and $S_{1.0}$ at T = 1.0 second. Where $S_{0.20}$ and $S_{1.0}$ are obtained as indicated in Steps 3 and 4, respectively.

After the Multi-Point horizontal ADRS curve has been constructed, the following should be checked to see if the Three-Point ADRS curve is underestimating spectral accelerations or not representative of the acceleration response spectrum.

- If fundamental periods of vibration greater than 1.0 second are important to the structural response, check Multi-Point spectral acceleration, S_a, corresponding to the 2.0 second period to assure that the long-period response is not underestimated.
- If fundamental periods of vibration less than 0.20 seconds are important to the structural response, check Multi-Point spectral acceleration, S_a, corresponding 0.10 sec period to assure that the short-period response is not underestimated.
- Check to see if the general trend of the Three-Point ADRS curve is similar to the Multi-Point ADRS curve. In certain circumstances there may be a shift that is not captured by the Three-Point ADRS, this is particularly true in the Eastern United States where the peak of the acceleration response spectrum is shifted towards the

1.0 second period. This shift appears to occur at project sites where the soil column is significantly deep and the site stiffness is $\overline{V}_S$ < 600 ft/sec. If the fundamental period of the structure is in the range of longer periods the spectral accelerations will be significantly underestimated using the Three-Point ADRS.

If discrepancies between the Three-Point method and the Multi-Point method have the potential to significantly underestimate the spectral response, the PCS/GDS must be contacted. The PCS/GDS will either approve modifications to the Three-Point ADRS curve or require a site-specific response analysis.

The ADRS curves in Figure 12-36 provide an example where discrepancies between the Three-Point method and the Multi-Point method indicate spectral accelerations (Sa) significantly underestimated at the 1.0 second period and significantly dissimilar acceleration response spectrum shape. The bridge location had a Site Class E and the fundamental period of the structure was 1.0 second. A site-specific response analysis was performed in accordance with Section 12.8 and the Site-Specific ADRS curve was generated for this example as shown in Figure 12-39.

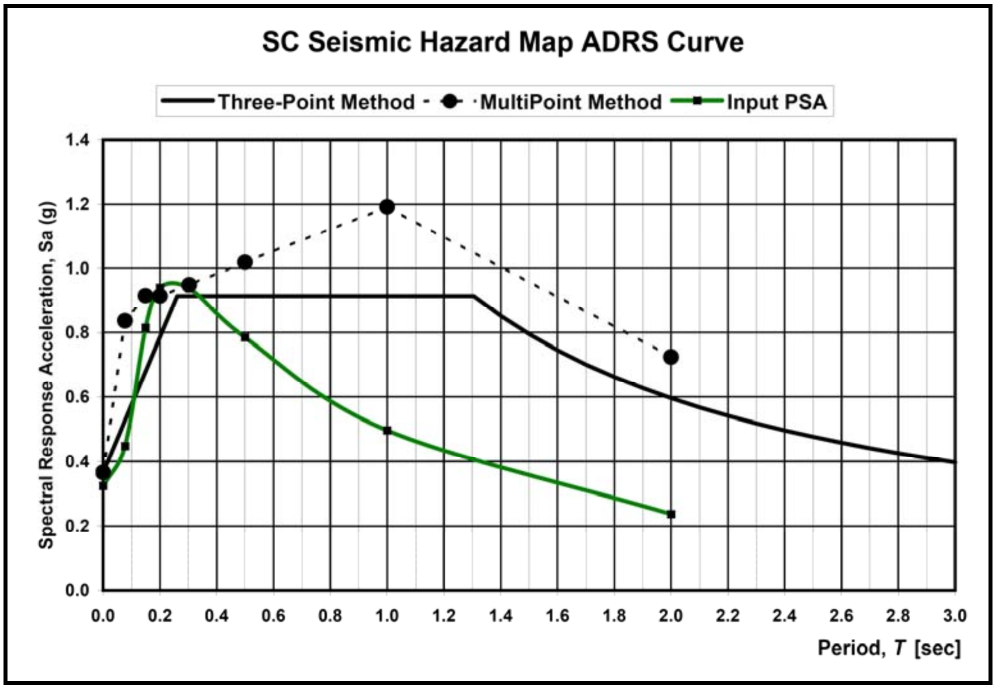


Figure 12-36, Three-Point and Multi-Point Method Comparison (Site Class=E)

12.7.6 <u>ADRS Evaluation using SC Seismic Hazard Maps</u>

Even though ADRS determination using SC Seismic Hazard maps is relatively straight forward, a series of checks are necessary to ensure its appropriateness. This involves using the Three-Point method as the basis of developing the ADRS curve and the Multi-Point method to confirm its validity. A decision flow chart is shown in Figure 12-37 to assist the designer with developing the ADRS curve based on SC Seismic Hazard map.

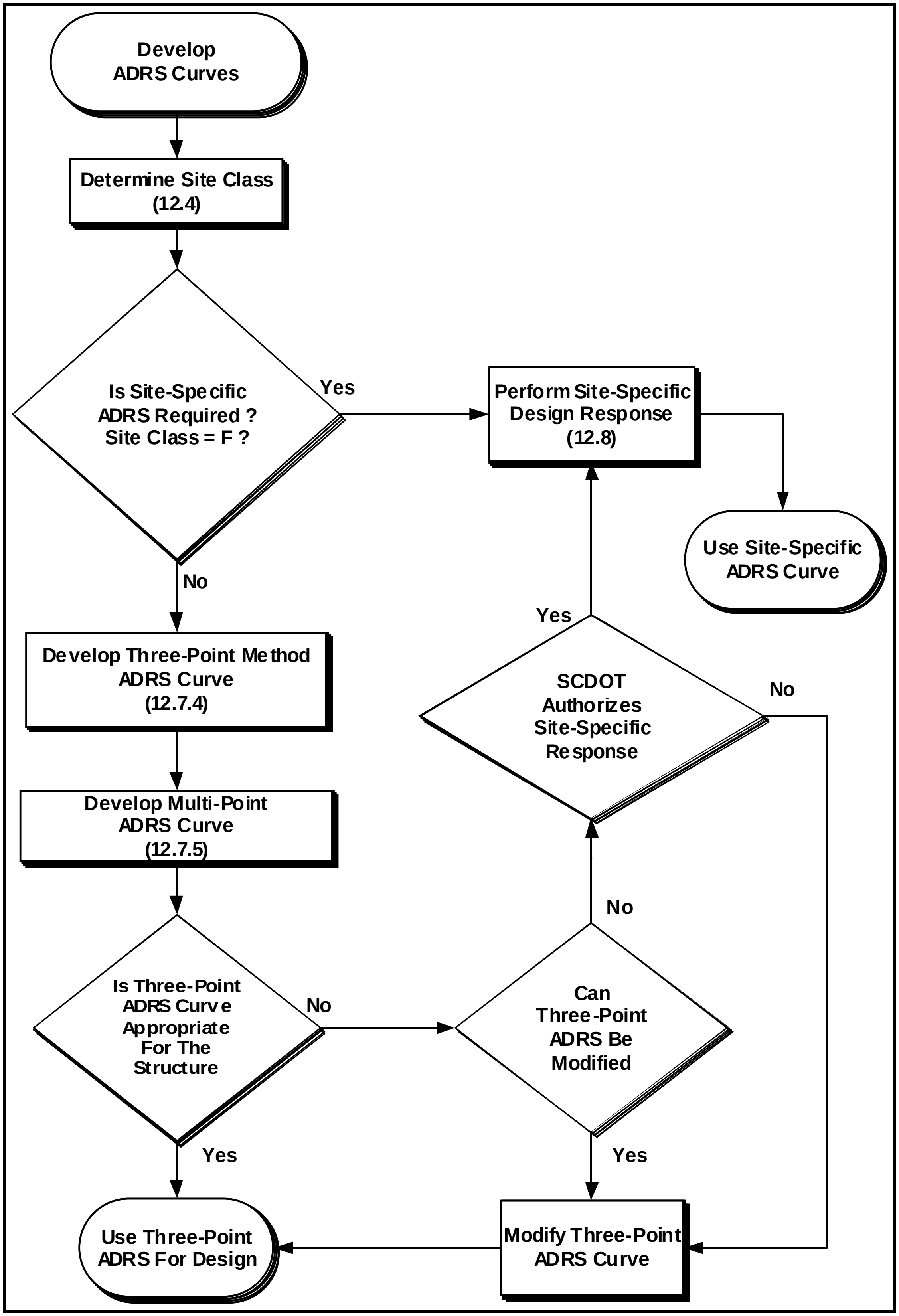


Figure 12-37, ADRS Curve Development Decision Chart

12.7.7 Damping Modifications of Horizontal ADRS Curves

The horizontal acceleration design response spectrum (ADRS) curves developed using the SC Seismic Hazard maps are based on a damping ratio of 5 percent. ADRS curves for structural damping ratios other than 5 percent can be obtained by multiplying the 5 percent damped ADRS curve by the period-dependent factors shown in Table 12-31. For spectra constructed using the Three-Point method, the factors for periods of 0.20 sec and 1.0 sec can be used.

Table 12-31, Damping Adjustment Factors
(Newmark and Hall, 1982, Abrahamson, 1993, and Idriss, 1993)

Period (seconds)	Ratio of Response Spectral Acceleration for Damping Ratio ξ to Response Spectral Acceleration for ξ_{eff} = 5%		
	ξ_{eff} = 2%	ξ_{eff} = 7%	ξ_{eff} = 10%
0.02	1.00	1.00	1.00
0.10	1.26	0.91	0.82
0.20	1.32	0.89	0.78
0.30	1.32	0.89	0.78
0.50	1.32	0.89	0.78
0.70	1.30	0.90	0.79
1.00	1.27	0.90	0.80
2.00	1.23	0.91	0.82
4.00	1.18	0.93	0.86

12.8 SITE-SPECIFIC RESPONSE ANALYSIS

The site-specific response analyses requirements in this section apply only to “Typical SCDOT Bridges” as defined by Sections 1.4 and 1.5 of the SCDOT *Seismic Design Specifications for Highway Bridges*. For non-typical bridges, the PCS/GDS will specify and/or approve appropriate geotechnical earthquake engineering provisions on a project specific basis. The site-specific response analysis is required when any of the following conditions are met.

- Structure has a Site Class F (Section 12.4)
- SC Seismic Hazard Maps are not appropriate (Section 12.7.5 and 12.7.6)
- As required by SCDOT

Site-specific ADRS curves that are generated using a non-linear effective stress site response software such as indicated in Sections 12.8.2 shall model the soils in both a liquefied and non-liquefied configuration and develop an ADRS envelope that combines the maximum spectral response amplifications for the site.

12.8.1 Equivalent-Linear One-Dimensional Site-Specific Response

An equivalent–linear one-dimensional site-specific response analysis shall be performed using SHAKE91 or other computer software that is based on the SHAKE91 computational model. The SHAKE91 computer program models a soil column with horizontal layered soil deposits overlying a uniform visco-elastic half space. The SHAKE91 computer program is based on the original SHAKE program developed by Schnabel, et al. (1972) and updated by Idriss and Sun

(1992). The computer program DeepSoil (Hashash et al., 2005) has been developed specifically for the CEUS and performs the equivalent linear analysis similar to Shake91. Requests to use software other than SHAKE91 or DeepSoil to perform the site-specific response analysis shall be made in writing to the PCS/GDS. Approval to use an alternate site-specific response analysis program shall be dependent on the software being nationally recognized in the United States as SHAKE91 type software and the designer is able to demonstrate project-specific experience using the proposed software.

For most projects and site conditions, the SHAKE91 method (or equivalent) of performing a site-specific response analysis will be required. When this method cannot accurately capture or model the site response, a non-linear one-dimensional effective stress site-specific response analysis may be required by the PCS/GDS. Situations where an equivalent–linear one-dimensional site-specific response analysis (SHAKE91) method has been shown to be unreliable are listed below:

- When ground-shaking levels are greater than 0.4g or if calculated peak shear strains exceed approximately 2 percent.
- When sites have significant liquefaction potential.
- When the non-linear mass participation factor (r_d) indicates either very low site stiffness, $V^*_{S,40'} <$ 400 ft/sec (120 m/sec) or very high site stiffness, $V^*_{S,40'} >$ 820 ft/sec (250 m/sec) and the project site has soil layers that have been screened to be potentially liquefiable.
- When seismic slope instability evaluations are required where complex geometries exist such as compound slopes, broken back slopes, or excessively high earth structures (embankments, dams, earth retaining systems).
- When sites have sensitive soils ($S_t > 8$).

12.8.2 <u>One-Dimensional Non-Linear Site-Specific Response</u>

The PCS/GDS must authorize the use of a non–linear one-dimensional effective stress site-specific response analysis. Guidance in using non-linear site response analysis procedures can be obtained from Kwok et al. (2007). One-dimensional non-linear site response analyses shall be performed using approved computer software such as DESRA-2 (Lee and Finn, 1978) that models the behavior of the soil subjected to cyclic loadings by tracing the evolution of the hysteresis loops generated in a soil by cyclic loading in a sequential manner. A number of other software programs such as D-MOD (Matasovick, 1993), DESRA-MUSC (Qiu, 1998), and DeepSoil (Hashash et al., 2005) have been developed that modify and improve the accuracy of the constitutive soil models originally developed. Authorized software used to perform one-dimensional non-linear site-specific response analysis must be based on the original DESRA-2 by Lee and Finn (1978) or equivalent. Requests to use software other than those indicated above to perform the non-linear site-specific response analysis shall be made in writing to the PCS/GDS. Approval to use an alternate non-linear site-specific response analysis program shall be dependent on the software being nationally recognized in the United States and the designer is able to demonstrate project-specific experience using the proposed software.

12.8.3 Earthquake Ground Motion

The SC Probabilistic Seismic Hazard study computer program Scenario_PC (2006) will be used to generate synthetic ground motions. The time histories generated by Scenario_PC (2006) are described in Section 11.8.4. The time history generated is a synthetic motion that can be matched to the uniform hazard or scaled to a period or frequency range of structural significance. Since a linear elastic time history dynamic analysis is being performed, a single time history matching the Uniform Hazard Spectrum will generally be sufficient for the majority of projects, particularly those located in the Coastal Plains. As indicated in Section 11.8.4, additional time histories may be needed based on the deaggregation results. Additional time histories may be required by SCDOT if project and site conditions warrant it.

12.8.4 Site Characterization

A one-dimensional soil column model is needed when performing a site-specific response analysis. The soil column extends from the location where the ground motion transmits the ground shaking energy to the structure being designed (depth-to-motion, Z_{DTM}, see Section 12.4.2) to the bedrock or geologically realistic site condition (B-C Boundary), where the ground motion has been developed.

When performing an equivalent–linear one-dimensional site-specific response analysis, the soil layers in the one-dimensional column are characterized by the Total Unit Weight (γ_{TW}), Shear Wave Velocity (V_s), Shear Modulus Reduction Curves (Normalized Shear Modulus, G/G_{max} vs. Shear Strain, γ), and Equivalent Viscous Damping Ratio Curves (Soil Damping Ratio, D vs. Shear Strain, γ). These soil parameters are described in Section 12.3. The soil column model should be prepared in tabular form similar to Table 12-32.

Table 12-32, One-Dimensional Soil Column Model

Geologic Time	Layer No.	Layer Thickness, H	Soil Formation	Soil Description (USCS)	*PI*	FC	Total Unit Weight, γ_{TW}	Shear Wave Velocity, V_s	Shear[(1)] Modulus Reduction Curve	Equivalent [(1)] Viscous Damping Ratio Curve
Quaternary	1									
	2									
Tertiary	3									
	4									
Cretaceous	5									
	6									
Bed Rock	i									

Note: PI = Plasticity Index; FC=% Passing the #200 sieve
(1) Indicate the cyclic stress-strain behavior method used by indicating reference (i.e. Andrus et al. (2003).

The development of the one-dimensional soil column for a project site may require making several assumptions as to the selection of layer thicknesses and soil properties. Therefore, the geotechnical engineer will need to perform a sensitivity analysis on the one-dimensional soil column model being developed to evaluate the consequences of the following:

- Variation in depth to B-C boundary and/or depth to basement rock
- Variations in soil properties for soils encountered below the maximum depth of the geotechnical investigation.

- Variations in soil properties of soils encountered during the geotechnical investigation across the project site.

The sensitivity analysis methodology must be well developed and documented in detail in the report. As a result of the sensitivity analysis performed, a series of site-specific horizontal acceleration response spectra (ARS) curves may be developed. A single recommended site-specific horizontal ARS curve should be superimposed on the graph. The method of selecting the recommended site-specific ARS curve should be documented in the report. The selection of the recommended site-specific ARS curve may be based on the sum of the squares (SRSS), the arithmetic mean, critical boundary method, or other method deemed appropriate. The method selected to develop the recommended site-specific ARS shall be indicated in the Site-Specific Response Analysis Study. The sensitivity analysis will be required for each time history developed for the project site.

When performing a non-linear one-dimensional effective stress site-specific response analysis the soil column model input motions shall be documented to at least the same level of detail as used in the equivalent-linear one-dimensional site-specific response analysis.

In addition to the site-specific design response report, all electronic input and output files shall be submitted to the PCS/GDS.

12.8.5 Site-Specific Horizontal ADRS Curve

The development of the recommended site-specific horizontal acceleration design response spectra (ADRS) shall be based on results of the site-specific response analysis (Sections 12.8.1 or 12.8.2). The Site-Specific ADRS curve should be developed for an equivalent viscous damping ratio of 5 percent. Additional ADRS curves may be required for other damping ratios appropriate to the indicated structural behavior. When the 5 percent damped Site-Specific ADRS curve has spectral accelerations in the period range of greatest significance to the structural response that are less than 70 percent of the spectral accelerations computed using the Three-Point Method, the PCS/GDS shall be consulted to determine if the spectral accelerations less than the 70 percent criteria can be used or if an independent third-party review of the ADRS curve by an individual with the expertise in the evaluation of ground motions is to be undertaken.

A smoothed Acceleration Design Response Spectrum (ADRS) curve shall be superimposed over the recommended site-specific acceleration response spectrum generated from site-specific response analysis (Sections 12.8.1 or 12.8.2). The steps to develop the smoothed ADRS curve shall be based on Table 12-33 and Figure 12-38.

Table 12-33, Site-Specific ADRS Construction Procedures

Step	Procedure Description
1	The maximum design spectral response acceleration, S_{DMax}, shall be taken as the spectral acceleration from the recommended site-specific acceleration response spectra at a period of 0.20 sec, except that it should not be taken as less than 90 percent of the peak response acceleration at any period.
2	With the plateau established as the value of S_{DMax} obtained from Step 1, graphically select value period markers, T_o and T_s, so as to create a best-fit of the site-specific response curve.
3	For spectral accelerations beyond the period of T_s, a smoothed curve based on Equation 12-44 shall be fitted over the site-specific acceleration response spectrum so that a best-fit is made with the site-specific response data so as not to allow any value to be less than 90 percent of the values obtained using the site-specific acceleration response spectrum. If the limitation of the 70 percent criteria of the Three-Point method is used as the lowest spectral acceleration permitted, the best-fit curve shall be adjusted to include the 70 percent criteria limitation. $$S_a = \frac{n}{e^T}$$ Equation 12-44 Where *T* is the period in seconds and *n* is a non-dimensional curve fitting number that is adjusted as required.
4	For periods, *T*, less than or equal to T_o, the design spectral response acceleration S_a shall be given by the following equation. $$S_a = PGA + \frac{T(S_{DMax} - PGA)}{T_o}$$ Equation 12-45 Where *PGA* is the spectral acceleration at a period, *T* = 0 seconds, S_{Dmax} is obtained from Step 1, and T_o is obtained from Step 2.
5	The site-specific response reports shall included the following items: • Recommended site-specific response curve • Smoothed Site-Specific ADRS curve • Table of smoothed ADRS data values (*T* and S_a) • Table with design spectral response parameters *PGA*, S_{Dmax}, S_{DS}, S_{D1}, and period markers T_o and T_s, as determined from the smoothed ADRS curve. • Equations 12-44 and 12-45 with all variables documented. An example of the information required is shown in Figure 12-39. The 2-per moving average ARS curve is used as an example, the recommended site-specific ARS curve should be constructed as indicated in Section 12.8.4.

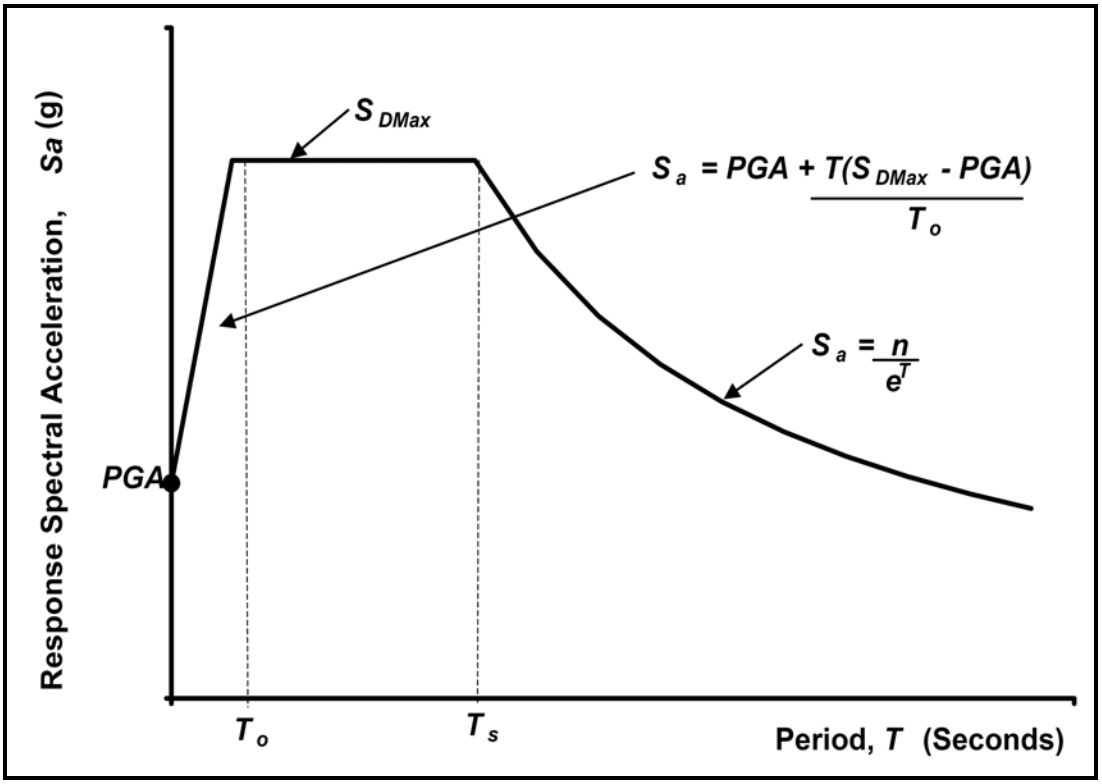
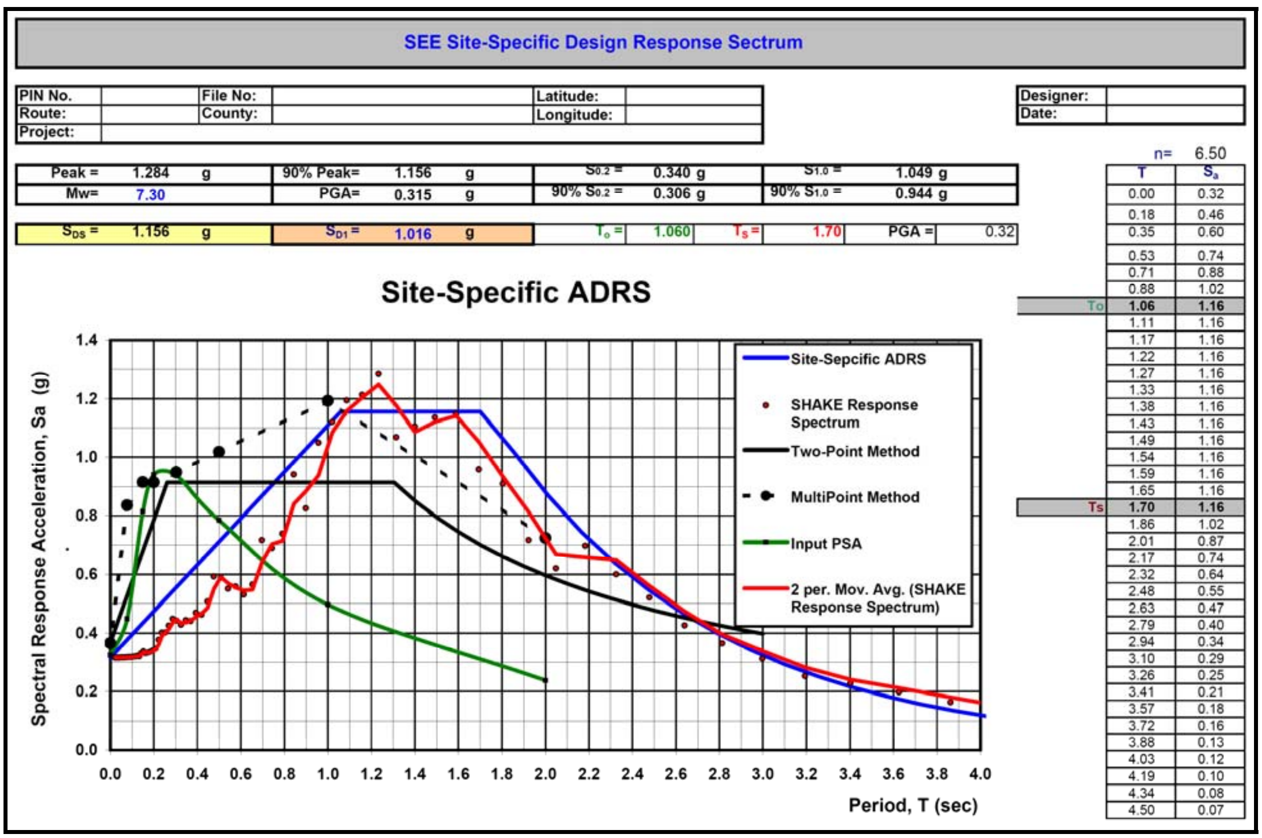


Figure 12-38, Site-Specific Horizontal ADRS Curve Construction



	T	S_a
	0.00	0.32
	0.18	0.46
	0.35	0.60
	0.53	0.74
	0.71	0.88
	0.88	1.02
To	**1.06**	**1.16**
	1.11	1.16
	1.17	1.16
	1.22	1.16
	1.27	1.16
	1.33	1.16
	1.38	1.16
	1.43	1.16
	1.49	1.16
	1.54	1.16
	1.59	1.16
	1.65	1.16
Ts	**1.70**	**1.16**
	1.86	1.02
	2.01	0.87
	2.17	0.74
	2.32	0.64
	2.48	0.55
	2.63	0.47
	2.79	0.40
	2.94	0.34
	3.10	0.29
	3.26	0.25
	3.41	0.21
	3.57	0.18
	3.72	0.16
	3.88	0.13
	4.03	0.12
	4.19	0.10
	4.34	0.08
	4.50	0.07

Figure 12-39, Site-Specific Horizontal ADRS Curve (Site Class E)

12.9 GROUND MOTION DESIGN PARAMETERS

12.9.1 Peak Horizontal Ground Acceleration

The peak horizontal ground acceleration (*PGA* or *PHGA*) at the ground surface is defined as the acceleration in the response spectrum obtained at a period, *T* = 0.0 seconds. If the Three-Point ADRS curves are used, the *PGA* obtained from Section 12.7.2 shall be used. If a site-specific response analysis is performed the spectral acceleration at period *T* = 0.0 second obtained from Site-Specific ADRS curve should be used.

12.9.2 Earthquake Magnitude / Site-to-Source Distance

The earthquake moment magnitude, M_W, and the site-to-source distance, R, can be obtained from the seismic hazard deaggregations charts discussed in Section 11.8.3.

12.9.3 Earthquake Duration

The earthquake duration is important when evaluating geotechnical seismic hazards that are influenced by degradation under cyclic loading. The longer the duration of the earthquake the more damage tends to occur. Geotechnical seismic hazards that would be affected by degradation under cyclic loading would be sites with cyclic liquefaction potential and liquefaction induced hazards such as lateral spreading and seismic instability.

The SCEC (Southern California Earthquake Center) DMG Special Publication 117 recommends using the Abrahamson and Silva (1996) relationship for rock. The Abrahamson and Silva (1996) correlation between moment magnitude (M_W), site-to-source distance (R), and the earthquake significant duration as a function of acceleration (D_{a5-95}) can be computed by the following equation.

R < 10 km:

Equation 12-46

$$\ln\left(\boldsymbol{D_{a5-95}}\right) = \ln\left[\frac{\left(\dfrac{\exp\left(\mathbf{5.204} + \mathbf{0.851}\left(\boldsymbol{M_W} - \mathbf{6}\right)\right)}{\mathbf{10}^{\left(\mathbf{1.5}\boldsymbol{M_W} + \mathbf{16.05}\right)}}\right)^{\left(-\frac{1}{3}\right)}}{\mathbf{15.7}\boldsymbol{x}\mathbf{10}^{6}}\right] + \mathbf{0.8664}$$

R ≥ 10 km:

Equation 12-47

$$\ln\left(\boldsymbol{D_{a5-95}}\right) = \ln\left[\frac{\left(\dfrac{\exp\left(\mathbf{5.204} + \mathbf{0.851}\left(\boldsymbol{M_W} - \mathbf{6}\right)\right)}{\mathbf{10}^{\left(\mathbf{1.5}\boldsymbol{M_W} + \mathbf{16.05}\right)}}\right)^{\left(-\frac{1}{3}\right)}}{\mathbf{15.7}\boldsymbol{x}\mathbf{10}^{6}} + \mathbf{0.063}\left(\boldsymbol{R} - \mathbf{10}\right)\right] + \mathbf{0.8664}$$

Where:

M_W	=	Moment magnitude of design earthquake (FEE or SEE) Section 12.9.2
R	=	Site-to-source distance (kilometers) Section 12.9.2

Kempton and Stewart (2006) developed a ground motion prediction equation to estimate the earthquake significant duration as a function of acceleration (D_{a5-95}) by using a modern database and a random-effects regression procedure. The correlation presented in the following equation uses the earthquake moment magnitude (M_W), site-to-source distance (R), site stiffness ($\overline{V}_S$ = $V_{S,30}$), and depth-to-hard rock (Z_{HR}) to estimate the earthquake significant duration (D_{a5-95}).

Equation 12-48

$$\ln\left(D_{a5-95}\right) = \ln\left[\frac{\left(\dfrac{\exp\left(2.79 + 0.82\left(M_W - 6\right)\right)}{10^{\left(1.5M_W + 16.05\right)}}\right)^{\left(-\frac{1}{3}\right)}}{15.68 \cdot 10^{6}} + 0.15R + 2.53 - 0.0041\overline{V}_S + 1.2 \cdot 10^{-3} Z_{HR}\right] + \varepsilon$$

Where:

$\overline{V}_S$	=	Site stiffness with Z_{DTM}=0 (Section 12.3.4)
M_W	=	Moment magnitude of design earthquake (FEE or SEE) Section 12.9.2
R	=	Site-to-source distance (kilometers) Section 12.9.2
Z_{HR}	=	Depth from ground surface to hard rock (Vs >5,000 ft/sec (1,500 m/s)) Units of
ε	=	Near-fault forward directivity correction for earthquakes (dip-slip or strike-slip faults)

R < 20 km: $\varepsilon = 0.015\left(R - 20\right)$

R ≥ 20 km: $\varepsilon = 0$

The Kempton and Stewart (2006) study confirmed the previous correlations (i.e. Abrahamson and Silva (1996)) that earthquake duration (D) increased with an increase in moment magnitude (M_W) and site-to-source distance (*R*). In addition, the study found that the earthquake duration (D) significantly increased with decreasing site stiffness ($\overline{V}_S$ = $V_{S,30}$). The earthquake duration (D) also increased slightly with an increase of depth-to-hard rock (Z_{HR}).

South Carolina shear wave profiles have indicate that site stiffness ($\overline{V}_S$ = $V_{S,30}$) can vary significantly across the state from a Site Class A(> 5,000 ft/s = 1,500 m/s) to a Site Class E (< 600 ft/s = 180 m/s). The effects of site stiffness on earthquake duration using Kempton and Stewart (2006) relationship have been plotted on Figure 12-40. An earthquake moment magnitude, M_W = 7.3 and a depth-to-hard rock, Z_{HR} = 2,600 feet (800m) have been selected as typical of the lower South Carolina Coastal Plain. The Abrahamson and Silva relationship for rock has also been plotted for reference.

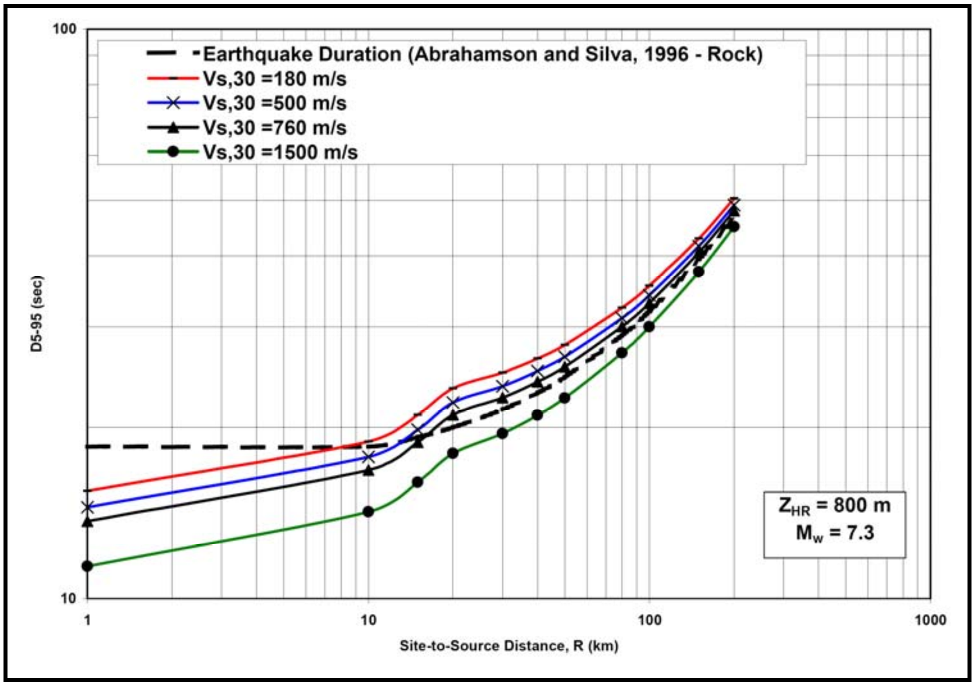


Figure 12-40, Effects of Site Stiffness on Earthquake Duration

South Carolina Coastal Plain geology (Chapter 11) indicates that the depth-to-hard rock varies from zero at the "Fall-line" up-to 4,000 feet (1,200 meters) at the southeastern corner of the state. The effects of depth-to-hard rock on earthquake duration using Kempton and Stewart (2006) relationship have been plotted on Figure 12-41. The Abrahamson and Silva relationship for rock has also been plotted as a reference.

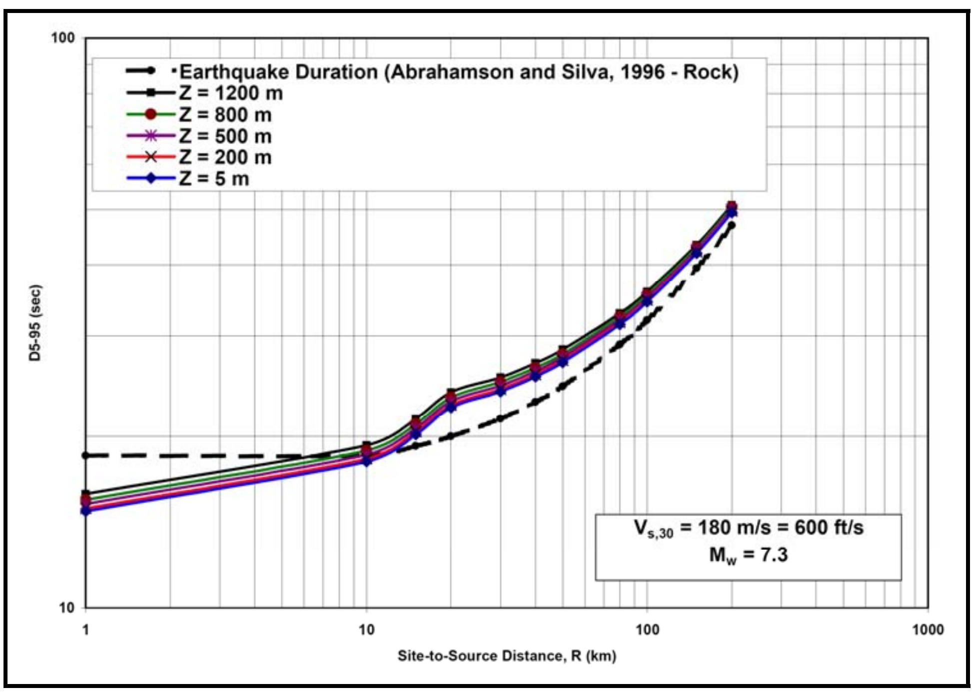


Figure 12-41, Effects of Depth-to-Hard Rock on Earthquake Duration

The project site conditions should be evaluated and the most appropriate earthquake duration model should be used.

12.9.4 Peak Ground Velocity

The peak ground velocity, V_{Peak}, of the earthquake can be determined from a site-specific response analysis. If the Three-Point ADRS curves are developed, peak ground velocity, V_{Peak}, correlations based on the NCHRP 12-70 document may be used.

The peak ground velocity, V_{Peak}, in units of in/sec can be computed by the following equation.

$$\boldsymbol{V_{Peak}} = \boldsymbol{PGV} = 55\boldsymbol{F_v S_1} \quad \text{Equation 12-46}$$

Where,

- F_v = site coefficient defined in Table 12-28, based on the Site Class and the mapped spectral acceleration for the long-period, S_1.
- S_1 = the mapped spectral acceleration for the one second period as determined in Sections 12.5 and 11.8.2 at the B-C boundary

12.10 REFERENCES

The geotechnical design specifications contained in this Manual must be used in conjunction with the *AASHTO* LRFD Bridge Design Specifications (latest edition). The SCDOT Seismic Design Specifications for Highway Bridges will take precedence over AASHTO seismic guidelines.

AASHTO (2007), "LRFD Bridge Design Specifications", 4th Edition, American Association of State Highway and Transportation Officials, Washington, D.C.

Abrahamson, N. A., and Silva, W. J., (1996), "Empirical Ground Motion Models", Report for Brookhaven National Laboratory, New York, NY, May, 144 pp.

Alpan, I., (1967), "The Empirical Evaluation of the Coefficients K_o and K_{or}", *Soils and Foundations*, Vol. 7, No. 1,31.

Andrus, R.D., Zhang, J., Ellis, B.S., and Juang, C.H., (2003), "Guide for Estimating the Dynamic Properties of South Carolina Soils for Ground Response Analysis", South Carolina Department of Transportation, SC-DOT Research Project No. 623, FHWA-SC-03-07.

Boore, D.M., (1983), "Stochastic simulation of high-frequency ground motions based on seismological models of the radiated spectra", Bulletin of the Seismological Society of America, v. 73, p. 1865-1894.

Brooker, E. W., and Ireland, H. O., (1965), "Earth Pressures at Rest Related to Stress History", Canadian Geotechnical Journal, vol. 2, no. 1, February, pp. 1-15.

Buckle, I., Friedland, I., Mander, J., Martin, G., Nutt, R., and Power, M. S., (2005), "Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges", Working Draft, MCEER. Buffalo, NY, December 2, 2005

Chapman M., (2008), Personal Communication - Lester, A.P., Thesis, “An Examination of Site Response in Columbia, South Carolina: Sensitivity of Site Response to “Rock” Input Motion and the Utility of Vs(30)”

Chiou, B.S., Silva, W. J., and Power, M.S., (2002), “Vertical to Horizontal Spectral Ratios for Seismic Design and Retrofit of Bridges in Western and Eastern United States”, Poster Session, Third National Seismic Conference and Workshop on Bridges and Highways, Portland, Oregon.

Darendeli, M. B., (2001), “Development of a New Family of Normalized Modulus Reduction and Material Damping Curves”, *Ph.D. Dissertation,* The University of Texas at Austin, Austin, TX.

Das, B. M., (1997), "Advanced Soil Mechanics”, Taylor & Francis, Washington, DC., 2nd Ed.

Dobry, R., Borcherdt, R. D., Crouse, C. B., Idriss, I. M., Joyner, W. B., Martin, G. R., Power, M. S., Rinne, E. E., and Seed, R. B., (2000), "New site coefficients and site classification system used in recent building seismic code provisions", Earthquake Spectra, 16(1), 41-67.

Dobry, R., Oweis, I., and Urzua, A., (1976), "Simplified Procedures for Estimating the Fundamental Period of a Soil Profile”, Bulletin of the Seismological Society of America, 66(4), ppl 293-1321.

Florence & Hutcheson (2006), “Draft Site-Specific Seismic Response Study US 378 Bridge Replacements over the Great Pee Dee River and the CSX Railroad, Florence and Marion Counties, South Carolina”, SCDOT File No. 21.182B.1 (PIN 30597), F&H Job No. 04052, Columbia, SC.

Green, R. A., (2001), "Energy-Based Evaluation and Remediation of Liquefiable Soils", Ph.D. Dissertation (J.K. Mitchell, Advisor), Virginia Polytechnic Institute and State University, 397pp.

Hadjian, A. and Green R. A., (2000), "Impact of Impedance and Resonance on Local Site Response", Proc., 6th International Conference on Seismic Zonation, Palm Springs, CA, Nov. 12-15, Earthquake Engineering Research Institute.

Hardin, B. O., (1978), "The Nature of Stress-Strain Behavior of Soils", Proc. Earthquake Engineering and Soil Dynamics”, ASCE, Pasadena, California, Vol. 1, pp. 3-89.

Hashash, Y., Park, D., Tsai, C., (2005), “DEEPSOIL Version 2.6 – Tutorial and User Manual”, University of Illinois at Urbana-Champaign, October 28, 2005.

Hunt, R. E., (1984), "Geotechnical Engineering Investigation Handbook”, Taylor & Francis, Boca Raton, Florida.

Idriss, I. M., (1990), "Response of Soft Soil Sites During Earthquakes", Proc. Memorial Symposium to Honor Professor H . B . Seed, Berkeley, California.

Idriss, I.M. and Sun, J.I., (1992), "User's Manual for SHAKE91", Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, California, 13 p. (plus Appendices).

Imai, T. and Tonouchi, K., (1982), "Correlation of N-Value with S-Wave Velocity and Shear Modulus", *Proc. 2nd European Symposium on Penetration Testing*, Amsterdam, The Netherlands, pp. 67-72.

Ishibashi, I., and Zhang, X. J., (1993), “Unified Dynamic Shear Moduli and Damping Ratios of Sand and Clay”, *Soils and Foundations,* Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 33, No. 1, 182-191.

Jaky, J., (1944), "The Coefficient of Earth Pressure at Rest”, Hungarian Architects and Engineers, Vol. 7, 355-358.

Jamiolkowski, M., Leroueil, S. and Lo Presti, D.C .F. (1991), "Theme Lecture: Design Parameters from Theory to Practice", Proc. Geo-Coast '91, Yokohama, Japan, pp. 1-41.

Kavazanjian, E., Jr., Matasovic, N., Hadj-Hamou, T., and Wang, J., (1998), "Geotechnical Earthquake Engineering”, Reference Manual, NHI Course No. 13239, U.S. Department of Transportation, Federal Highway Administration, National Highway Institute, Arlington, Virginia.

Kempton, J. J., and Stewart, P.S., (2006), "Prediction equations for significant duration of earthquake ground motions considering site and near-source effects", Earthquake Spectra, 22(4), 985-1013.

Kwok, A.O.L., Stewart, J.P., Hashash, Y.M.A., Matasovic, N., Pyke, R., Wang, Z., Yang, Z., (2007), "Use of Exact Solutions of Wave Propagation Problems to Guide Implementation of Nonlinear Seismic Ground Response Analysis Procedures", ASCE Journal of Geotechnical and Geoenvironmental Engineering Division, Volume 133, Issue 11, pp.1385-1398.

Lee, M.K. W. and Finn, W .D.L. (1978), "DESRA-2, Dynamic Effective Stress Response Analysis of Soil Deposits with Energy Transmitting Boundary Including Assessment of Liquefaction Potential", Soil Mechanics Series No. **36,** Department of Civil Engineering, University of British Columbia, Vancouver, Canada, 60 p.

Lewis, M. R., McHood, M. D., and Arango, I., (2004), “Liquefaction Evaluation at the Savannah River Site – A Case History”, Proceedings*: Fifth International Conference on Case Histories in Geotechnical Engineering*, New York, NY, April 13-17, 2004, Paper No. 3.21.

Madera, G. A., (1970), “Fundamental Period and Amplification of Peak Acceleration in Layered Systems”, Research Report R70-37, Soils Publication No. 260, Department of Civil Engineering, MIT, 77pp.

Matasovic, N., (1993), "Seismic Response of Composite Horizontally-Layered Soil Deposits", Ph.D. Dissertation, Civil and Environmental Engineering Department, University of California, Los Angeles, 452 p.

Mayne, P.W., Brown, D., Vinson, J., Schneider, J.A., and Finke, K.A., (2000), "Site characterization of Piedmont residual soils at the NGES, Opelika, Alabama", *National Geotechnical Experimentation Sites,* (GSP No. 93), American Society of Civil Engineers, Reston/VA, 160-185.

Mayne, P. W., and Kulhawy, F. H., (1982), “Ko-OCR relationships in soil”, ASCE Journal of the Geotechnical Engineering Division, Vol. 108, No. GT6, pp. 851-872.

Mayne, P. W. and Rix, G. J., (1993), "G_{max}-q_c, Relationships for Clays", Geotechnical Testing Journal, ASTM, Vol. 16, No. 1, pp. 54-60.

NCHRP 12-70 (2007), “Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankments”. NCHRP, CH2M Hill, November, 2007.

Odum, J.K., Williams, R.A., Stepheson, W.J., and Worley, D.M., (2003), “Near-surface S-wave and P-wave seismic velocities of primary geological formations on the Piedmont and Atlantic Coastal Plain of South Carolina, USA”, United States Geological Survey Open-File Report 03-043, 14p.

Oweis, I., Urzua, A., and Dobry, R., (1975), “Simplified Procedures for Estimating the Fundamental Period of a Soil Profile”, Proceedings: Thirteenth Annual Engineering Geology and Soils Engineering Symposium, 2-4 April, Moscow, ID, pp51-58.

Power, M., Fishman, K., Richards, R., Makdisi, F., Musser, S., and Youd, T. L., (2004), “Seismic Retrofitting Manual for Highway Structures: Part 2 – Retaining Structures, Slopes, Tunnels, Culverts, and Pavements”, Working Draft, MCEER. Buffalo, NY, August 2004.

Power, M.S., and Chiou, S.-J., (2000), *National Representation of Seismic Ground Motion for New and Existing Highway Facilities,* Unpublished Report, Multidisciplinary Center for Earthquake Engineering Research, University at Buffalo.

Power, M.S., Chiou, S.-J., Rosidi, D., and Mayes, R.L., (1997), “Background Information for Issue A: Should New USGS Maps Provide a Basis for the National Seismic Hazard Portrayal for Highway Facilities? If So, How Should They be Implemented in Terms of Design Values?”, *Proceedings of the FHWA/NCEER Workshop on the National Representation of Seismic Ground Motion for New and Existing Highway Facilities*, Burlingame, California, May 29-30, Technical Report NCEER-97-0010, National Center for Earthquake Engineering Research, University at Buffalo.

Power, M.S., Mayes, R.L., and Friedland, I.M., (1998), “National Representation of Seismic Ground Motion for New and Existing Highway Facilities”, *Proceedings of Sixth National Conference on Earthquake Engineering*, Earthquake Engineering Research Institute, May 31-June 4, Seattle, Washington.

Power, M.S., Rosidi, D., Kaneshiro, J., Gilstrap, S.D., and Chiou, S.J., (1998), *Summary and Evaluation of Procedures for the Seismic Design of Tunnels,* Draft Report, Multidisciplinary Center for Earthquake Engineering Research, September.

Qiu, P., (1998), Earthquake-induced Nonlinear Ground Deformation Analyses, Ph.D. dissertation, University of Southern California, Los Angeles.

Robertson, P. K., (1990). “Soil Classification Using the Cone Penetration Test”, *Canadian Geotechnical Journal,* Vol. 27, No. 1, 151-158.

Robertson, P. K., and Wride (Fear), C. E., (1998). “Cyclic Liquefaction and Its Evaluation Based on the SPT and CPT”, *Proceedings of the National Center for Earthquake Engineering Research (NCEER) Workshop on Evaluation of Liquefaction Resistance of Soils*, Salt Lake City, Utah, January 1996, Edited by T. L. Youd and I. M. Idriss, NCEER Report NCEER-97-0022, 41-87.

Rodriquez-Marek, Bray, J. D. , and Abrahmson, N. A., (1999), “Task 3: Characterization of site response, general site categories”, Rpt. No. PEER 1999/03, Pacific Earthquake Engineering Research Center, Richmond, CA.

S&ME (2000), “Phase II Geotechnical Data Summary Report – Cooper River Bridge Replacement Project, Charleston, South Carolina”, S&ME Job No. 1131-97-741, Mount Pleasant, SC.

S&ME (2007), “Site Specific Seismic Study – Wetland Bridges 1, 2, 3, & 4”, Charleston, South Carolina, U.S. Route 17 – Design Build, Beaufort County, South Carolina”, SCDOT File No. 7.412B, S&ME Job No. 1131-07-065, Mount Pleasant, SC.

Sadigh, R.K. and Egan, J.A., (1998), “Updated Relationships for Horizontal Peak Ground Velocity and Peak Ground Displacement for Shallow Crustal Earthquakes”, *Proceedings of the Sixth U.S. National Conference on Earthquake Engineering*, Seattle, Washington.

SCDOT (2007), “Seismic Design Specifications for Highway Bridges”, South Carolina Department of Transportation

Schmertmann, J. H., (1975), "The Measurement of In-Situ Shear Strength”, 7th ASCE PSC, Vol. 2, pp. 57-138.

Schnabel, P. B., Lysmer, J. and Seed, H. B., (1972), "SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites", Report No. EERC 72-12, Earthquake Engineering Research Center, University of California, Berkeley, California.

Seed, H. B., and Idriss, I. M., (1970), “Soil Moduli and Damping Factors for Dynamic Response Analysis”, *Report EERC 70-10,* Earthquake Engineering Research Institute, Berkeley, CA.

Seed, H. B., and Idriss, I. M., (1982), “Ground Motions and Soil Liquefaction During Earthquakes”, *EERI Monograph,* Earthquake Engineering Research Institute, Berkeley, CA.

Seed, H.B., Ugas, C., and Lysmer, J., (1976), “Site-dependent spectra for earthquake resistant design”, Bulletin of the Seismological Society of America, v. 66, p. 221-243.

Seed. H.B., Wong, R.T., Idriss, I.M. and Tokimatsu, K., (1984), "Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils", Report No. UCBIEERC-84/14, Earthquake Engineering Research Center, University of California, Berkeley, California.

Seed, H. B., Wong, R. T., Idirss, I. M., and Tokimatsu, K., (1986), “Moduli and Damping Factors for Dynamic Analysis of Cohesionless Soils”, *Journal of the Geotechnical Engineering Division,* ASCE, Vol. 112, No. 11, 1016-1031.

Silva, W.J. and Darragh, R., (1995), “Engineering characterization of earthquake strong ground motion recorded at rock sites”, Electric Power Research Institute, TR-102261.

Silva, W.J. and Green, R.K., (1989), “Magnitude and distance scaling of response spectral shapes for rock sites with applications to North American tectonic environment”, Earthquake Spectra, v. 5, p. 591-624.

Stokoe, K. H., II, Hwang, S. K., Darendeli, M. B., and Lee, N. J., (1995), “Correlation Study of Nonlinear Dynamic Soils Properties”, final report to Westinghouse Savannah River Company*,* The University of Texas at Austin, Austin, TX.

Stokoe, K. H., II, Darendeli, M. B., Andrus, R. D., and Brown, L. T., (1999). “Dynamic Soil Properties: Laboratory, Field and Correlation Studies”, *Proceedings, 2nd International Conference on Earthquake Geotechnical Engineering,* Vol. 3, Lisbon, Portugal, 811-845.

Sykora, D. W., (1987). “Examination of Existing Shear Wave Velocity and Shear Modulus Correlation in Soils”, *Miscellaneous Paper GL-87-22,* U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

URS Corporation (2001), “Comprehensive Seismic Risk and Vulnerability Study for the State of South Carolina”, South Carolina Emergency Management Division (SCEMD).

Vucetic, M., and Dobry, R., (1991), “Dynamic Effect of Soil Plasticity on Cyclic Response”, *Journal of Geotechnical Engineering*, ASCE, Vol. 117, No. 1, 89-107.

Youd, T. L., and Carter, B.L., (2005), "Influence of Soil Softening and Liquefaction on Spectral Acceleration". ASCE Journal of Geotechnical and Geoenvironmental Engineering Division, Volume 131, Issue 7, pp.811-825.

Chapter 13

GEOTECHNICAL SEISMIC HAZARDS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Chapter under Construction

Chapter 14

GEOTECHNICAL SEISMIC DESIGN

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Chapter under Construction

Chapter 15

SHALLOW FOUNDATIONS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Chapter under Construction

Chapter 16

DEEP FOUNDATIONS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Chapter under Construction

Chapter 17

STABILITY AND SETTLEMENT ANALYSIS AND DESIGN

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Chapter under Construction

Chapter 18

EARTH RETAINING STRUCTURES

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Chapter under Construction

Chapter 19

GROUND IMPROVEMENT

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Chapter under Construction

Chapter 20

GEOSYNTHETIC DESIGN

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Chapter under Construction

Chapter 21

GEOTECHNICAL REPORTS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Chapter under Construction

Chapter 22

PLAN PREPARATION

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Chapter under Construction

Chapter 23

SPECIFICATIONS AND SPECIAL PROVISIONS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Chapter under Construction

Chapter 24

CONSTRUCTION QA/QC

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Chapter under Construction

Chapter 25

CONSTRUCTION INSPECTION AND INSTRUMENTATION

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Chapter under Construction

Chapter 26

GEOTECHNICAL SOFTWARE

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Chapter under Construction

Appendix A Index

Geotechnical Scoping Form

PROJECT INFORMATION		
File No.	PIN:	Date of Trip:
County:	Location:	
Rd/Route:	Local Name:	
Charge Code:	Track:	
Attendees:		

EXISTING BRIDGE INFORMATION		
Bridge Length:	Bridge Width:	
Superstructure Type:	Substructure Type:	
Begin Sta.:	End Sta.:	
Structure Number:	Crossing:	Posted Weight Limit:
Latitude:	Longitude:	

EXISTING SITE INFORMATION
Accessibility Issues:
Ground Cover:
Local Development (undeveloped, developed residential, developed commercial, developed industrial etc.):
Topography (level, flat, rolling, steep, hillside, valley, swamp, gully, etc.):
Traffic Control Necessary (Y/N):

HYDRAULICS INFORMATION		
Surface Soil:	Muck (Y/N):	Skew:
Exposed Rock (Y/N):	In Stream Bed (Y/N):	In Banks (Y/N):
Wetlands On-Site (Y/N):	Wetlands Adjacent (Y/N):	
Depth FG to Water:	Water Depth:	
Depth to Existing Ground:	Flow:	
Scour Condition at EB:	Scour Condition at IB:	

UTILITIES INFORMATION
Attached:
Above Ground/ Overhead:
Underground:

COMMENTS

*******Optional Diagram, Additional Boring Information on Back*******

Bridge Load Data Sheet

PROJECT INFORMATION	
File No.	Project No. (PIN):
County:	Route:
Description:	
Report Request By:	Date Requested:

BRIDGE STRUCTURE INFORMATION			
Bridge Type:			
No. Spans /Lengths:		**Width / No. Lanes:**	
Bridge Category / Seismic OC:			
Seismic Performance Category (SPC):			
Seismic Site Class:			
Structural Design Method:	LRFD ☐	LFD ☐	

Proposed Foundations (foundation type, size, and number per bent)

End Bent	
Interior Bent	

HYDRAULICS INFORMATION

Design Scour	Contraction Scour (feet)	Local Scour (feet)	Total Scour (feet)
100 Yr			
500 Yr			

BRIDGE LOADS

Location/Elev. of Applied Loads:	End Bent:		Int. Bent:	
Location/Elev. Est. Point of Fixity:	End Bent:		Int. Bent:	

End Bent Foundation Loads

Strength Axial Loads (kips):	DL		DL + LL	

Interior Bent Foundation Loads

(Strength I, II, III, IV, and V) Longitudinal Loads (Along the bridge or perpendicular to bent cap)

Load Cases:	**Case 1FL ($P=P_{max}$)**		**Case 2FL ($V=V_{max}$)**		**Case 3FL ($M=M_{max}$)**	
P (axial - kips) =	DL+ LL	DL	DL+ LL	DL	DL+ LL	DL
V (shear - kips) =						
M (moment – ft-kip) =						

(Strength I, II, III, IV, and V) Transverse Loads (Transverse to the bridge or in direction bent cap)

Load Cases:	Case 1FT ($P=P_{max}$)		Case 2FT ($V=V_{max}$)		Case 3FT ($M=M_{max}$)	
P (axial - kips) =	DL+ LL	DL	DL+ LL	DL	DL+ LL	DL
V (shear - kips) =						
M (moment – ft-kip) =						

End Bent Foundation Loads

Seismic Performance ***(Required for SPC = B, C, D)***

Extreme Event I

Load Cases:	Maximum Axial Load ($P=P_{max}$)
P (axial - kips) =	

Interior Bent Foundation Loads

Seismic Performance ***(Required for SPC = B, C, D)***

Extreme Event I

Load Cases:	Maximum Axial Load ($P=P_{max}$)
P (axial - kips) =	

Consultant Seismic Information Request

PROJECT INFORMATION			
File No.			**Project No. (PIN):**
County:	**RPG[1]:**		**Route:**
Description:			
Latitude (4 decimals): .			**Longitude (4 decimals):** .

SEISMIC REQUEST

The SCDOT Geotechnical Design Manual and Seismic Design Specifications for Highway Bridges, latest editions, provide detailed seismic design requirements for transportation structures. The RPG Geotechnical Design Section (GDS) will be generating seismic design information from, *SCENARIO_PC*, the seismic analysis software. The consultant is encouraged to review the software documentation, *Information on Analysis Software*, for assistance in completing this form. The RPG GDS will be providing the pseudo-spectral acceleration (PSA) oscillator response for frequencies 0.5, 1.0, 2.0, 3.3, 5.0, 6.7 and 13 Hz, for 5% critical damping and peak horizontal ground acceleration (PGA) at either the **B-C Boundary** (Geologically Realistic) or **Hard Rock** Outcrop for specific project locations within South Carolina. The Geologically Realistic option is for sites in the Coastal Plain with sediment thickness greater than 100 feet to firm sediment (V_s=2,500 feet per second (ft/s) or NEHRP B-C Boundary). Geologically Realistic conditions can also be encountered outside of the Coastal Plain where the sediment thickness is 100 feet or less above the basement rock and the V_s = 8,000 ft/s. The Hard Rock Outcrop option is for an outcrop of hard rock ($V_s \geq$ 11,500 ft/s). The Preconstruction Support – Geotechnical Design Section (PCS/GDS) has developed a map to assist in determining the site condition. South Carolina has been divided in two zones, Zone **I** – Physiographic Units Outside of the Coastal Plain and Zone II – Physiographic Units of the Coastal Plain. This information can be provided for the Safety Evaluation Earthquake (SEE) 3% probability of exceedance for 75-year exposure periods or for the Functional Evaluation Earthquake (FEE) 15% probability of exceedance for 75-year exposure periods. The consultant is reminded that all embankment structures are required to be designed for both the SEE and FEE. The consultant will use this information in developing the Acceleration Design Response Spectrum (ADRS) in accordance with the SCDOT Geotechnical Design Manual and Seismic Design Specifications for Highway Bridges. The RPG GDS can also provide the Time Series for use in performing a Site-Specific Response Analysis.

STRUCTURE SEISMIC INFORMATION

Bridge Category / Seismic OC:	
Seismic Performance Category (SPC):	
Seismic Site Class:	
Bridge Seismic Level of Design:	

Select Design Earthquake

SEE – 3% Probability of Exceedance in 75 years	☐
FEE – 15% Probability of Exceedance in 75 years	☐

Geologically Realistic ☐ **Hard Rock Basement Outcrop** ☐

Requestor Information

Requestor Name:	
Company Name:	
Phone Number:	( ) -
Email Address	
Request Date:	

Consultant Seismic Information Request

PROJECT INFORMATION	
File No.	**Project No. (PIN):**

TIME SERIES GENERATION REQUEST

Time Series information is required if a Site-Specific Response Analysis is to be conducted. The SCDOT Geotechnical Design Manual requires a Site-Specific Response Analysis for Seismic Site Class "F". Unscaled and Scaled time series will be generated for the **B-C Boundary** in Shake91 data format. The Scaled time series are based on the earthquake magnitude (Mw) and Epicentral distance provided.

Request Time Series: Yes ☐ No ☐

Sediment Thickness

The sediment thickness is used by *SCENARIO_PC*, to generate the time series simulation. The time series can be generated with the default sediment thickness as indicated in *2.2.2.1 Site Response Modeling* of the *Seismicity Study Report* (http://www.scdot.org/doing/pdfs/Reporttxt.pdf) or can adjusted specifically for the geology and analysis requirements at the specific project location. This option only applies to those site were the Geologically Realistic Model is used.

Change Sediment Thickness: Yes meters **No** ☐

Match Entire Uniform Spectrum

In cases where the uniform hazard spectrum is dominated by a single scenario (a well defined modal event in the Deaggregation plots), the spectrum of the modal event may closely match that of the uniform hazard spectrum, even without much scaling. This will be the case for sites in the Coastal Plain near Charleston, for the 3% in 75 year hazard level. However, at sites where there are two or maybe 3 modes in the deaggregation, matching the entire spectrum with a single modal event will require much scaling. This scaling can be done automatically over the entire spectrum. Matching the entire spectrum involves a phase-invariant spectral scaling of the scenario time series. It is often preferable to use two or more modal events, each matching a specific frequency of the uniform hazard spectrum. This results in a simple constant (frequency independent) scaling of the scenario time series. If the consultant selects to not match the entire spectrum, the spectrum may be scaled using either an oscillator frequency/PSA or a PGA that will be matched when simulating the ground motion.

Match Entire Spectrum:	**Yes** ☐	**No** ☐		
		Scaling Parameter	**M_{w1}**	**M_{w2}**
If Not matching Entire Spectrum, Select PSA or PGA Scaling	**PSA Scaling** ☐	**Oscillator Frequency**	**Hertz**	**Hertz**
		PSA	**g**	**g**
	PGA Scaling ☐	**PGA**	**g**	**g**

Scenario Earthquake Magnitude and Distance

Determine earthquake magnitude, M_W, and epicentral distance from the deaggregation plots provided by the U.S. Geological Survey (http://eqint.cr.usgs.gov/deaggint/2002/index.php). The 3% and 15% in 75-year events are equivalent to the 2% and 10% in 50-year events, respectively.

M_{W1} =	**Epicentral Distance =**	Kilometers
M_{W2} =	**Epicentral Distance =**	Kilometers

[1]**RPG – Region Production Group**

- **Lowcountry** - Beaufort, Berkeley, Charleston, Colleton, Dorchester, Hampton, Jasper
- **Pee Dee** – Chesterfield, Clarendon, Darlington, Dillon, Florence, Georgetown, Horry, Kershaw, Lee, Marion, Marlboro, Sumter, Williamsburg
- **Midlands** – Aiken, Allendale, Bamberg, Barnwell, Calhoun, Chester, Fairfield, Lancaster, Lexington, Newberry, Orangeburg, Richland, Union, York
- **Upstate** – Abbeville, Anderson, Cherokee, Edgefield, Greenville, Greenwood, Laurens, McCormick, Oconee, Pickens, Saluda, Spartanburg

Consultant Geotechnical Seismic Response

To:	
Consultant:	
Date Requested:	

PROJECT INFORMATION

File No.	**Project No. (PIN):**
County:	**Route:**
Description:	
Latitude (4 decimals): .	**Longitude (4 decimals):** .

Bridge Category / Seismic OC:	
Type of Seismic Information Requested:	
Seismic Site Class:	

Pseudo-Spectral Acceleration (PSA)

The SCDOT Geotechnical Design Section has generated the required Design Earthquake the pseudo-spectral acceleration (PSA) oscillator response for frequencies 0.5, 1.0, 2.0, 3.3, 5.0, 6.7 and 13 Hz, for 5% critical damping and peak horizontal ground acceleration (PGA) at the **B-C Boundary.**

SEE – 3% Probability of Exceedance in 75 years

PSA and PGA as Percentage of g

0.5Hz	1.0Hz	2.0Hz	3.3Hz	5.0Hz	6.7Hz	13.0Hz	PGA

Thickness of sediments: meters

FEE – 15% Probability of Exceedance in 75 years

PSA and PGA as Percentage of g

0.5Hz	1.0Hz	2.0Hz	3.3Hz	5.0Hz	6.7Hz	13.0Hz	PGA

Thickness of sediments: meters

Time Series

Unscaled and Scaled time series were generated for the **B-C Boundary** in Shake91 data format. The Scaled time series are based on the earthquake magnitude (Mw) and Epicentral distance requested.

The Time Series Files are Attached:	**Yes** ☐	**No** ☐

Design Response Spectrum

The SCDOT Seismic Design Specifications for Highway Bridges, latest edition, is used to develop the Design Response Spectrum.

The Design Response Spectrum is Attached:	**Yes** ☐	**No** ☐

Geotechnical Designer:		**RPG[1]:**	
Date:		**Phone Number:**	( ) -
Geotechnical Review:		**RPG[1,2]:**	

[1]**RPG – Region Production Group**

Lowcountry - Beaufort, Berkeley, Charleston, Colleton, Dorchester, Hampton, Jasper

Pee Dee – Chesterfield, Clarendon, Darlington, Dillon, Florence, Georgetown, Horry, Kershaw, Lee, Marion, Marlboro, Sumter, Williamsburg

Midlands – Aiken, Allendale, Bamberg, Barnwell, Calhoun, Chester, Fairfield, Lancaster, Lexington, Newberry, Orangeburg, Richland, Union, York

Upstate – Abbeville, Anderson, Cherokee, Edgefield, Greenville, Greenwood, Laurens, McCormick, Oconee, Pickens, Saluda, Spartanburg

[2]**RPG – PreConstruction Support – Geotechnical Design Section (PCS/GDS)**

South Carolina
Department of Transportation

INTEROFFICE MEMORANDUM

To: Director of Rights-of-Way

From: RPG

Date:

Subject: Access Permission Request

The following project is being prepared for Geotechnical Subsurface Investigation:

County:
Road:
File:
Project No.:
PIN No.:
Location:
Project Name:
Charge Code:
Project Manager:

Project Management has provided us with plans, and we will visit the above referenced site in the coming weeks. Based upon the information provided, we understand the following design concepts are under consideration at this time:

- The proposed bridge will be constructed on the existing horizontal alignment.
- The grade will be raised approximately XX ft above the existing finish grade elevation
- This project will encompass approximately.

Roadway and Bridge borings will need to be performed between Stations XX+XX to XX+XX on Anywhere Road, some of which are on SCDOT Right-of-Way and others that are not. Installation of an accessway will be required for this project. This may entail removal of some trees using heavy equipment to permit access. It may also be necessary for us to bring in fill soil to bridge soft, wet areas. Every effort will be made by the Contractor to minimize damage to property and as few trees as possible will be disturbed in the process. Below is a table of anticipated boring locations for the project site. It must be pointed out that the boring locations are planned and may change if site conditions warrant or utilities such as overhead power lines necessitate relocation of the proposed borings.

Table 1 (Road)

Boring No.	Road Cut (C)/ Road Fill (F)	Proposed Stationing	Offset Distance (ft)*	Boring Depth (ft.)

***Offset from construction centerline, both left and right**

Table 2 (Bridge)

Boring No.	Proposed Stationing	Offset Distance (ft)*

***Offset from construction centerline, both left and right**

South Carolina
Department of Transportation

Attached are the Geotechnical Design Section's Scoping forms (Form GDF 000), one (1) full-sized set and one (1) half-sized set of plans depicting the proposed soil test boring locations for the project. Bridge and roadway soil borings will be required as indicated on the plans.

We anticipate the access permission to be available by Month day, Year so we can begin mobilizing the drillings. Once signed permission has been obtained, please provide a copy of the signed document to us. We will provide a copy of this document to the drillers, who will be required to maintain copies physically in their possession at all times during drilling operations.

If you have any questions or comments, feel free to contact either Jeff Sizemore at (803) 737-1571 or, Sara Stone at (803) 737-1608. Or you can email me at **StoneSM@scdot.org**.

Sara M. Stone
Geotechnical Professional

Jeff Sizemore, P.E.
Geotechnical Design Engineer

JCS/SMS: xxx
cc: BDF, Project Management, Geotech file

South Carolina
Department of Transportation

Date:

To:

Re: File No. , PIN

County

If you have any comments or questions, please contact us.

Your Name
Your Title

cc:

South Carolina
Department of Transportation

Date: March 10, 2005

To: Consultant

From: RPG

Re: Soil Exploration Testing and Compressive Strength Testing of Rock Cores

Soil Exploration and Testing of soil samples and Compressive strength testing of rock core samples is requested for the following project

County:
Road:
Route Local Name:
File:
Project No.:
PIN No.:
Location:
Project Name:
Charge Code:
Priority: ***Lab test information needed April 22, 2005.***
Final Boring Logs needed April 29, 2005.

Boring Number	Sample Depth (ft)	Sample Number	Grain Size with wash #200	Atterberg Limit	Natural Moisture Content
B-1	0 - 2				
	2 – 4				
	4 – 6				
	8 - 10				
	13.5 – 15.0				
	18.5 – 20.0				
	23.5 – 25.0				
	28.5 – 30.0				
	33.5 – 35.0				
	43.5 – 45.0				
B-2	0 – 2				
	2 – 4				
	4 – 6				
	6 – 8				
	8 - 10				
	18.5 – 20.0				
	23.5 – 25.0				
	38.5 – 40.0				
B-3	22.0 – 24.0				
	24.0 – 26.0				
	26.0 – 28.0				
	28.0 – 30.0				
	30.0 – 32.0				
	48.5 – 50.0				

Note: ** Conduct hydrometer analysis also.

Boring Number	Recovery (%)	RQD(%)	Core Number	Number of Breaks Requested
B-2				
B-3				
B-4				
B-5				
B-6				

Please e-mail an electronic copy and forward a hard copy of the results to Sara Stone so that the information can be included in the contract document. If you require any additional information, please contact Sara Stone at 737-1608.

Requested by:

Sara Stone
Geotechnical Professional

cc: BDF, Geotech