Geotechnical Design Manual

June 2010

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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 August 2008, Version 1.0, and the June 2010 Chapters 13 through 26 and Appendices B through G join together to form the complete SCDOT Geotechnical Design Manual, Version 1.1. Any questions concerning the applicability of procedure, analysis, or method should be directed to the PCS/GDS for review and comment. As modifications arise, the modifications will be issued in accordance with established SCDOT guidelines.

[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."*

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Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

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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.

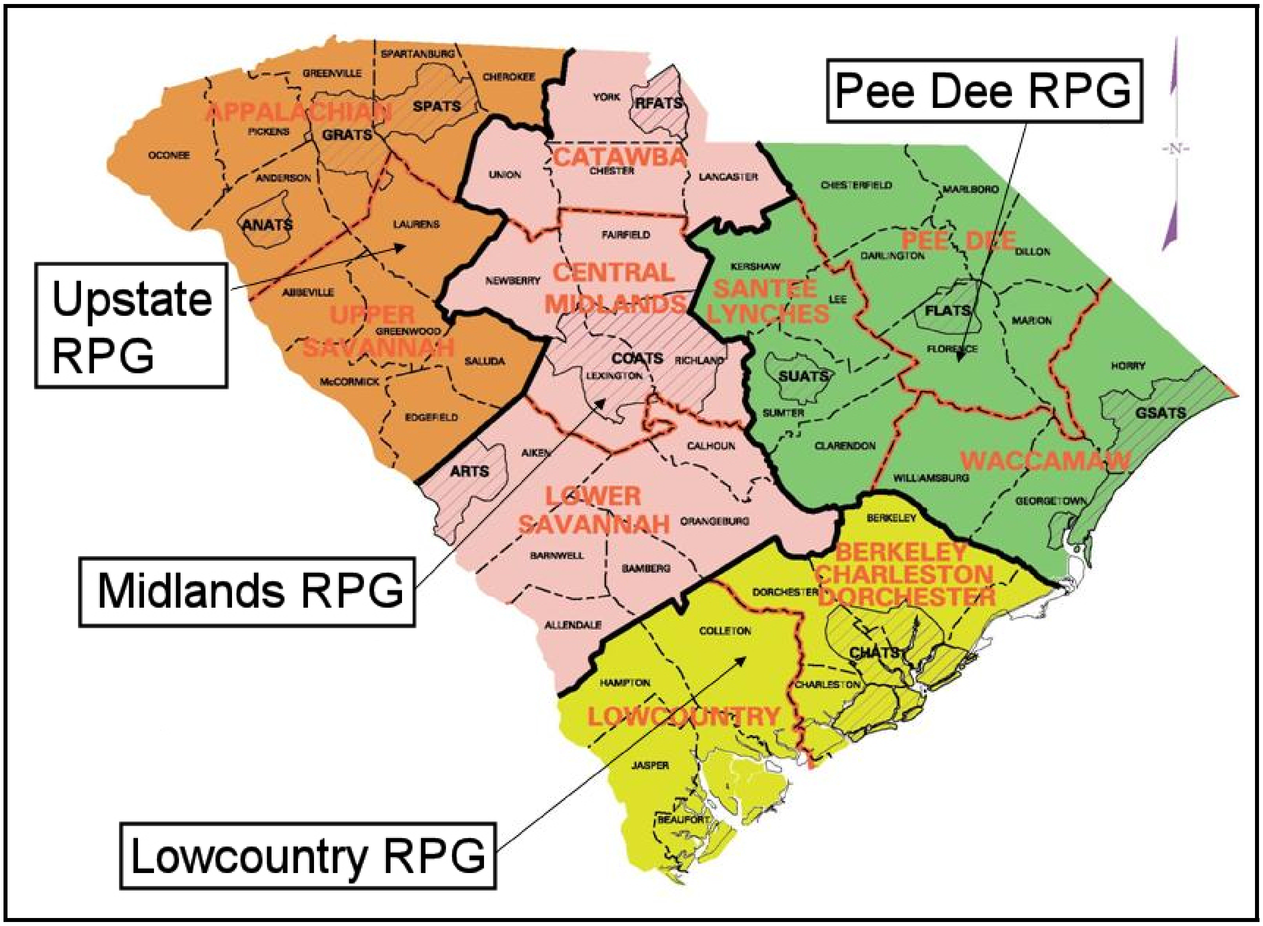

Figure 1-1, Regional Production Groups

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 Traffic Engineering Division

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 Construction Division

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. Shop Plans. 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. Installation Plans. 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

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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.

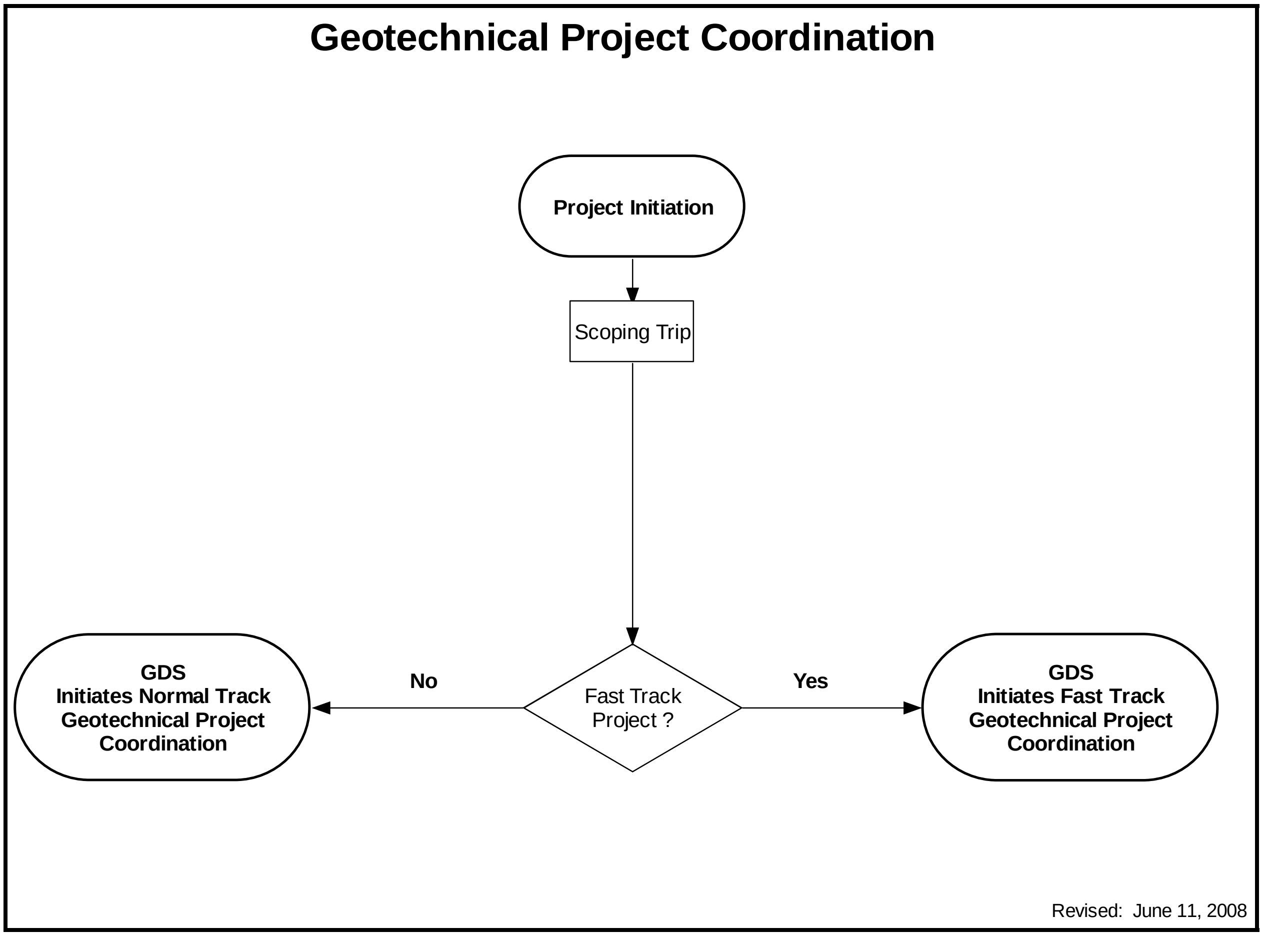

Figure 2-1, Project Initiation Process

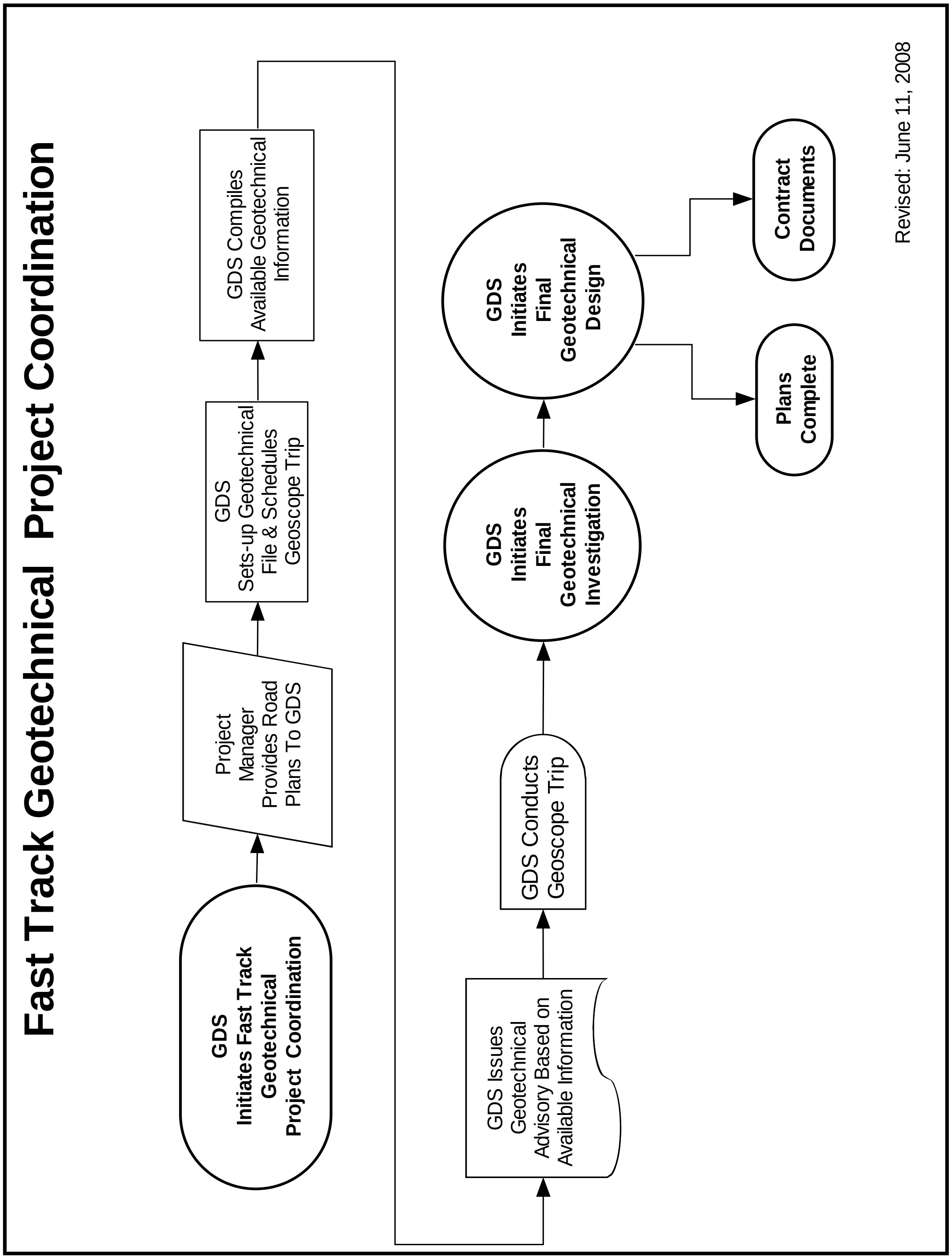

Figure 2-2, Fast Track Geotechnical Project Coordination

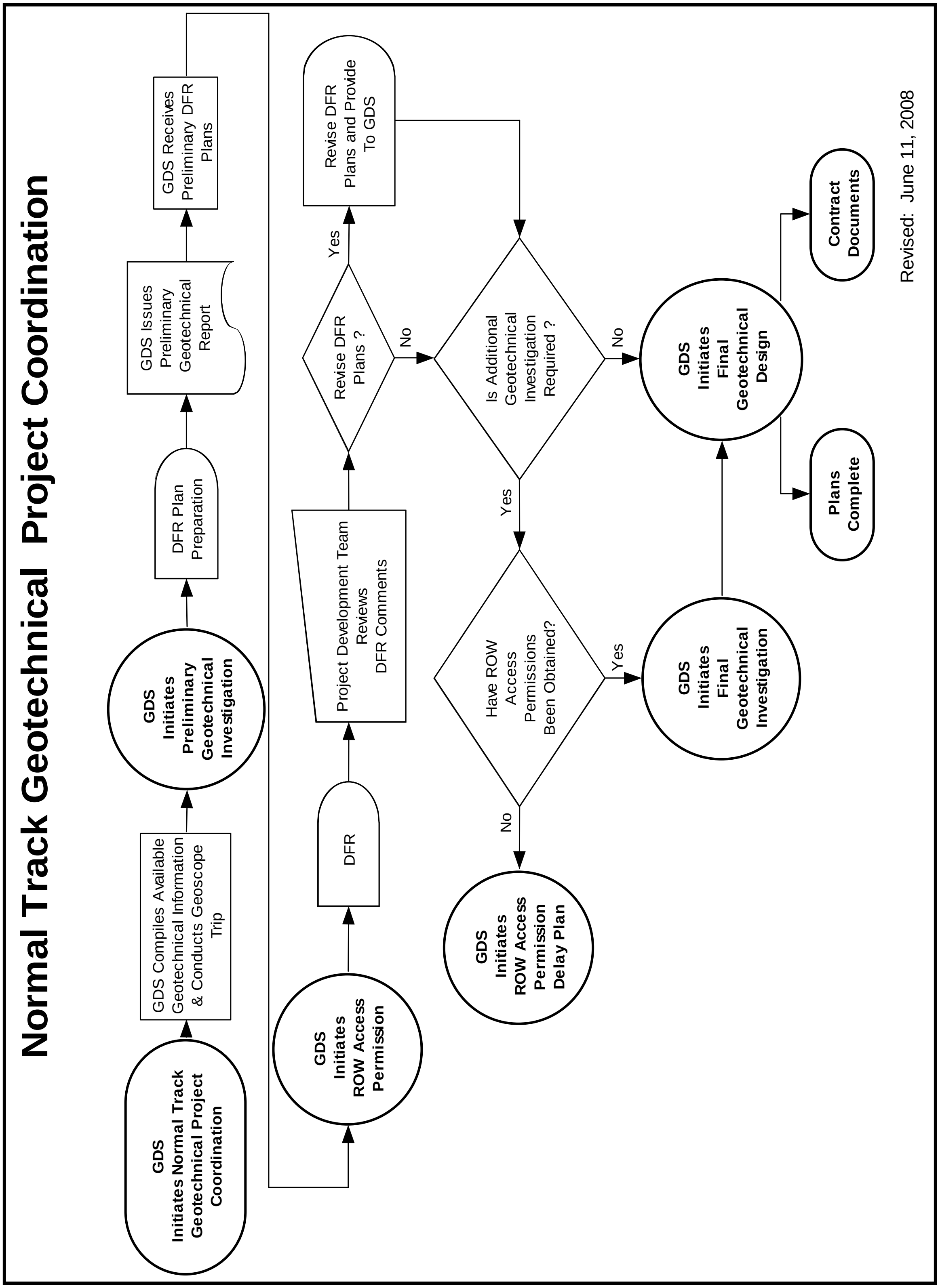

Figure 2-3, Normal Track Geotechnical Project Coordination

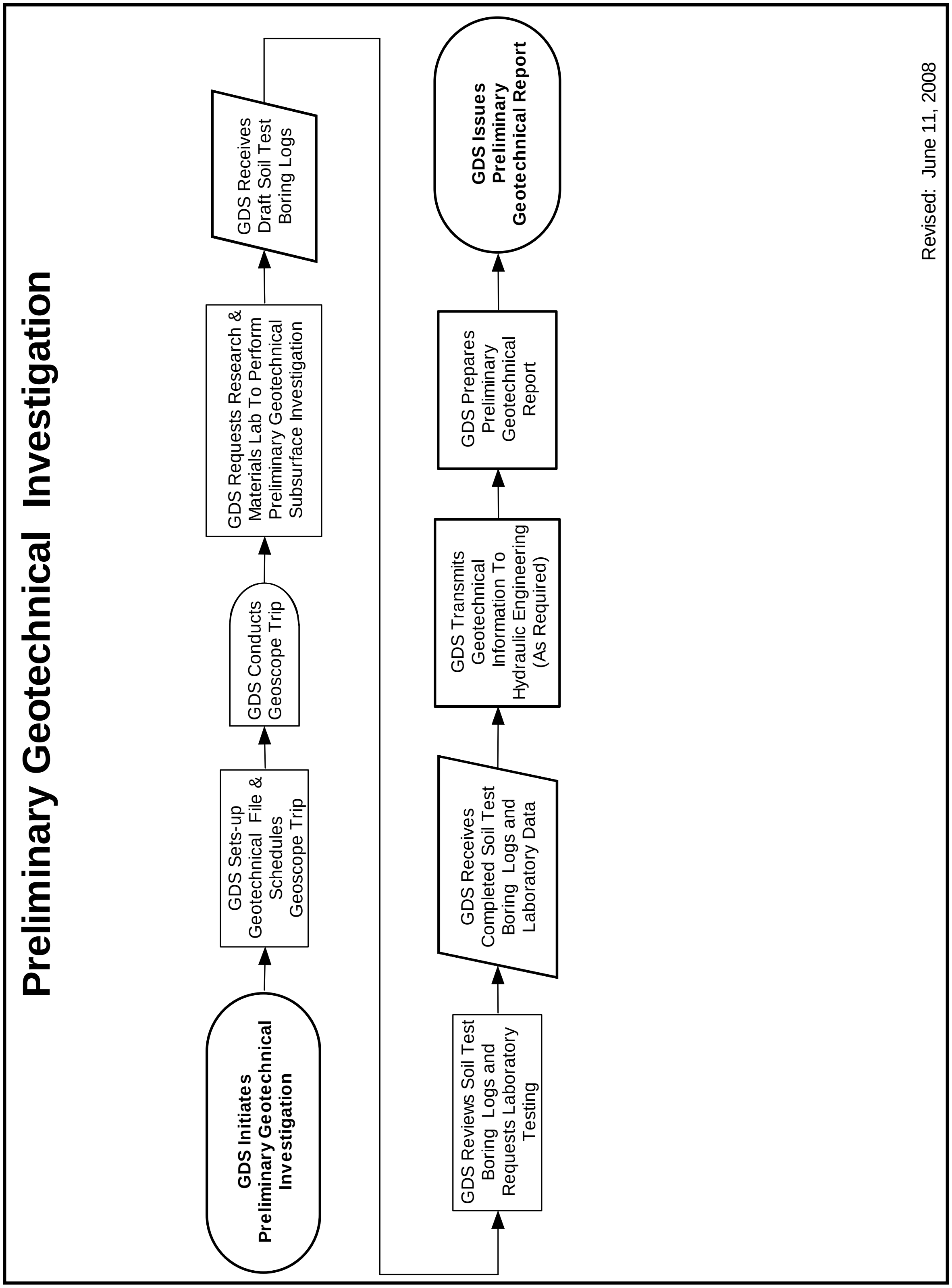

Figure 2-4, Preliminary Geotechnical Investigation

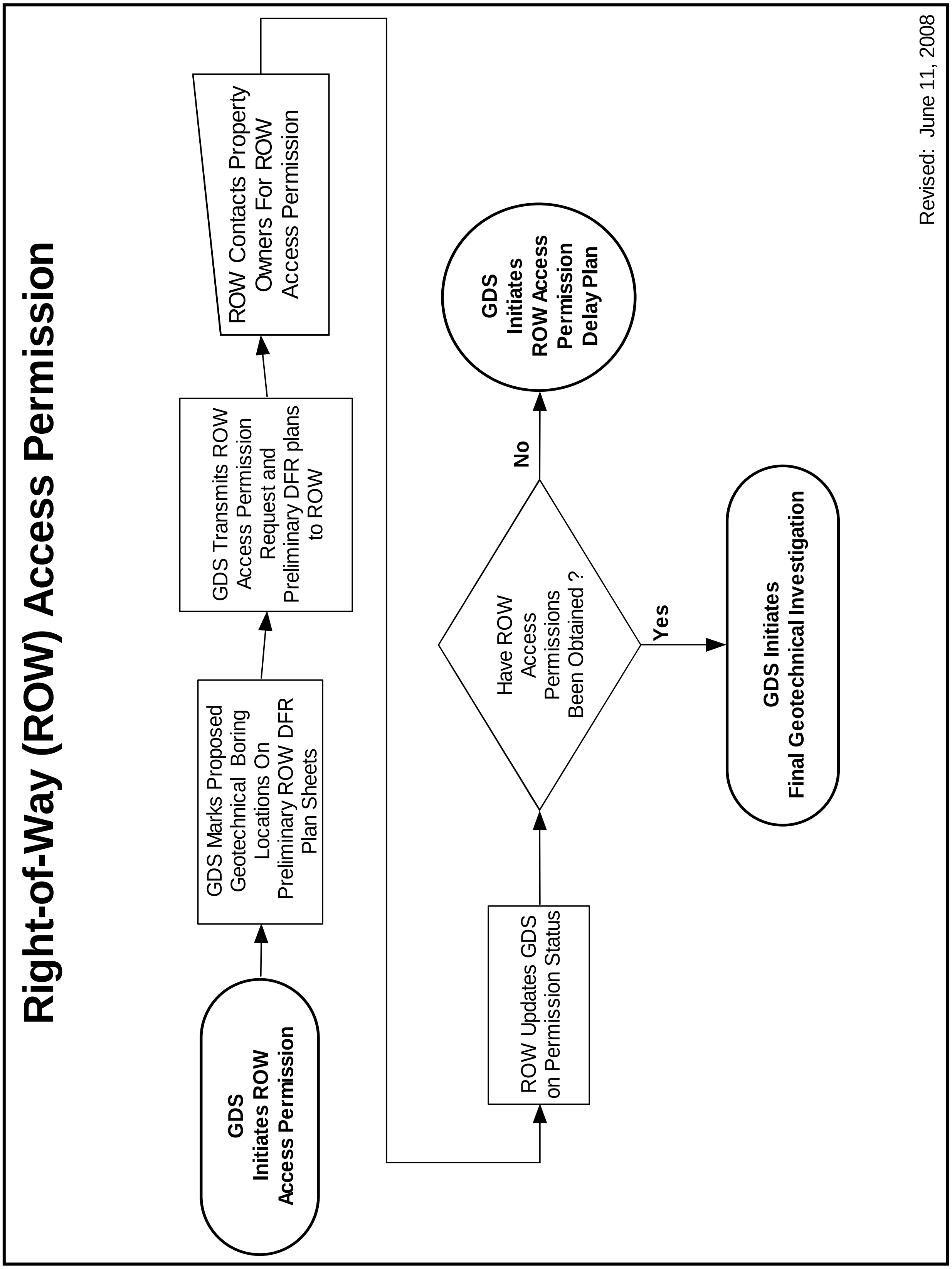

Figure 2-5, Right-of-Way Access Permission

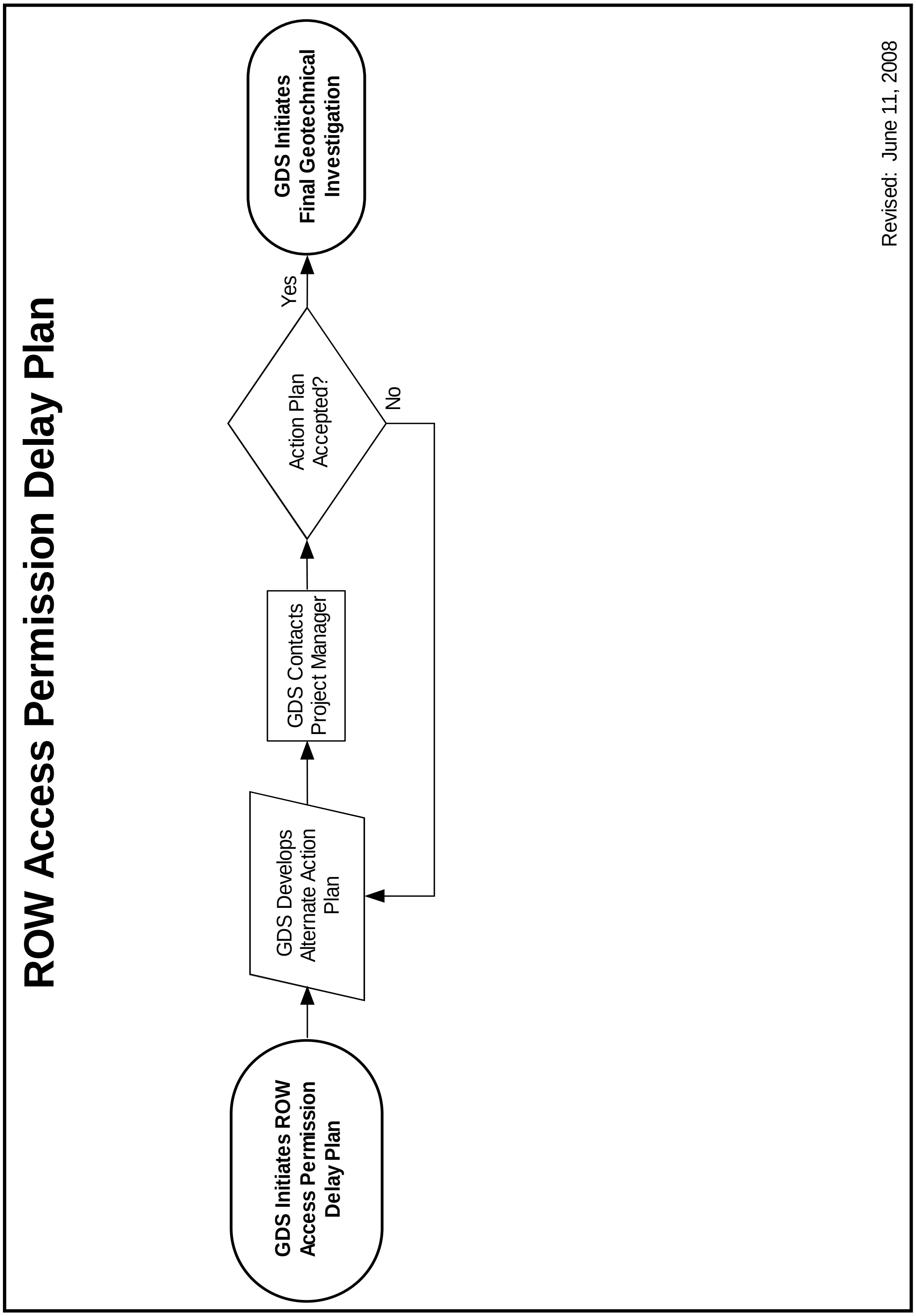

Figure 2-6, Right-of-Way Access Permission Delay Plan

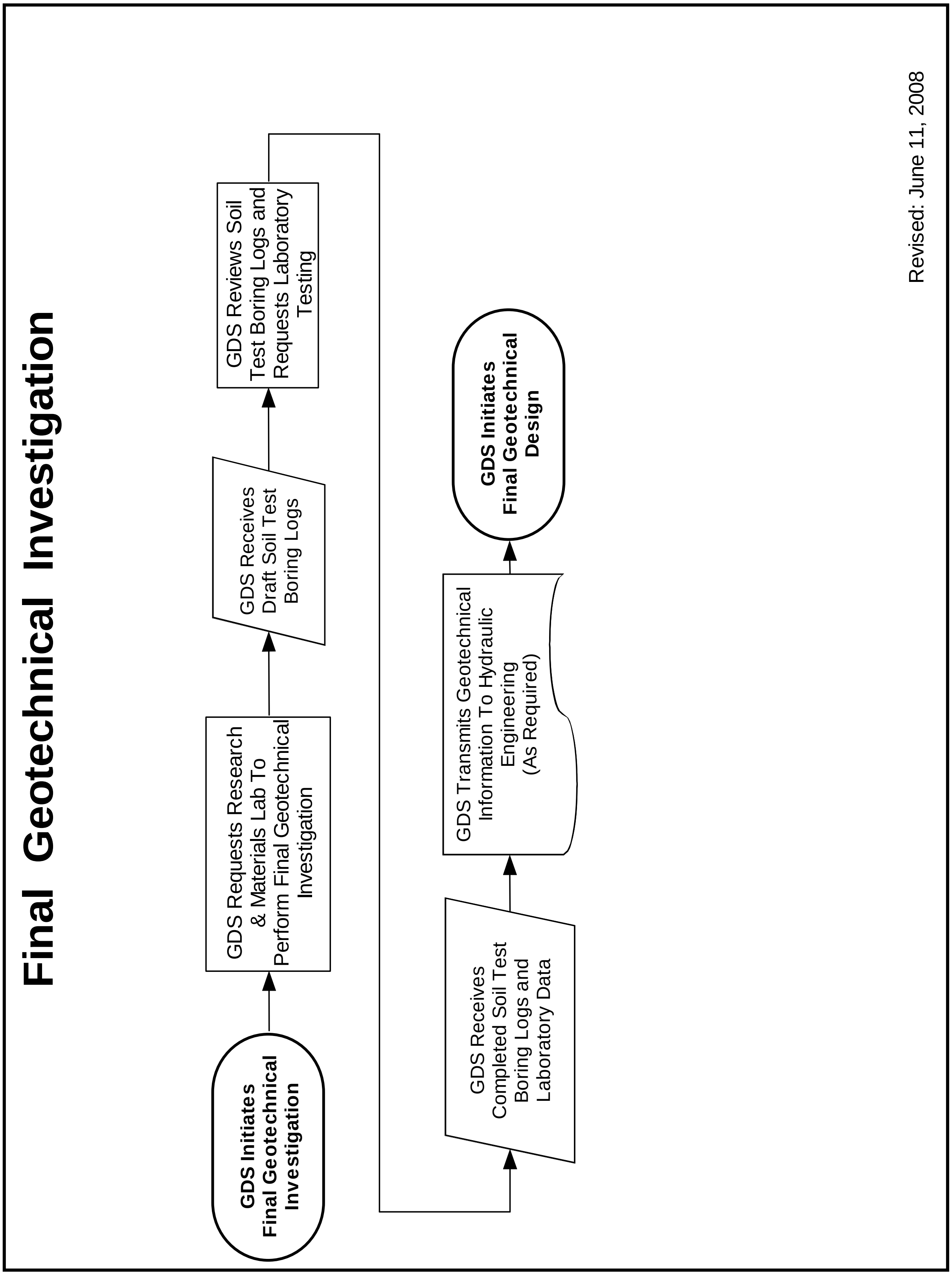

Figure 2-7, Final Geotechnical Investigation

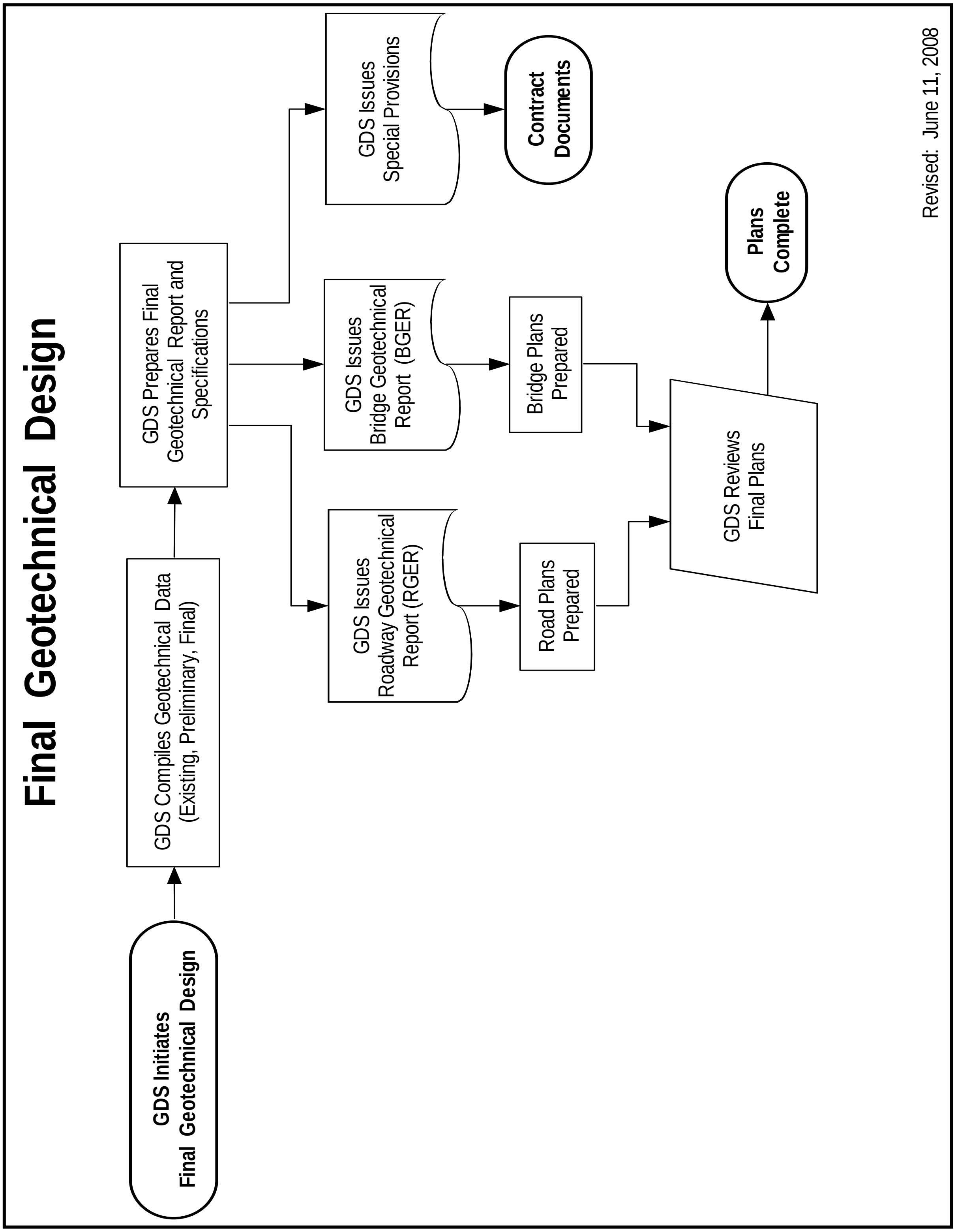

Figure 2-8, Final Geotechnical Design

Chapter 3

CONSULTANT SERVICES AND REVIEW

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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

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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 Preliminary Subsurface Investigation

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

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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 <u>Soil Sampling</u>

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 Soils*).

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 Geophysical Testing Methods

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 Grain-Size Analysis

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'.

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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

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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).

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

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

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.

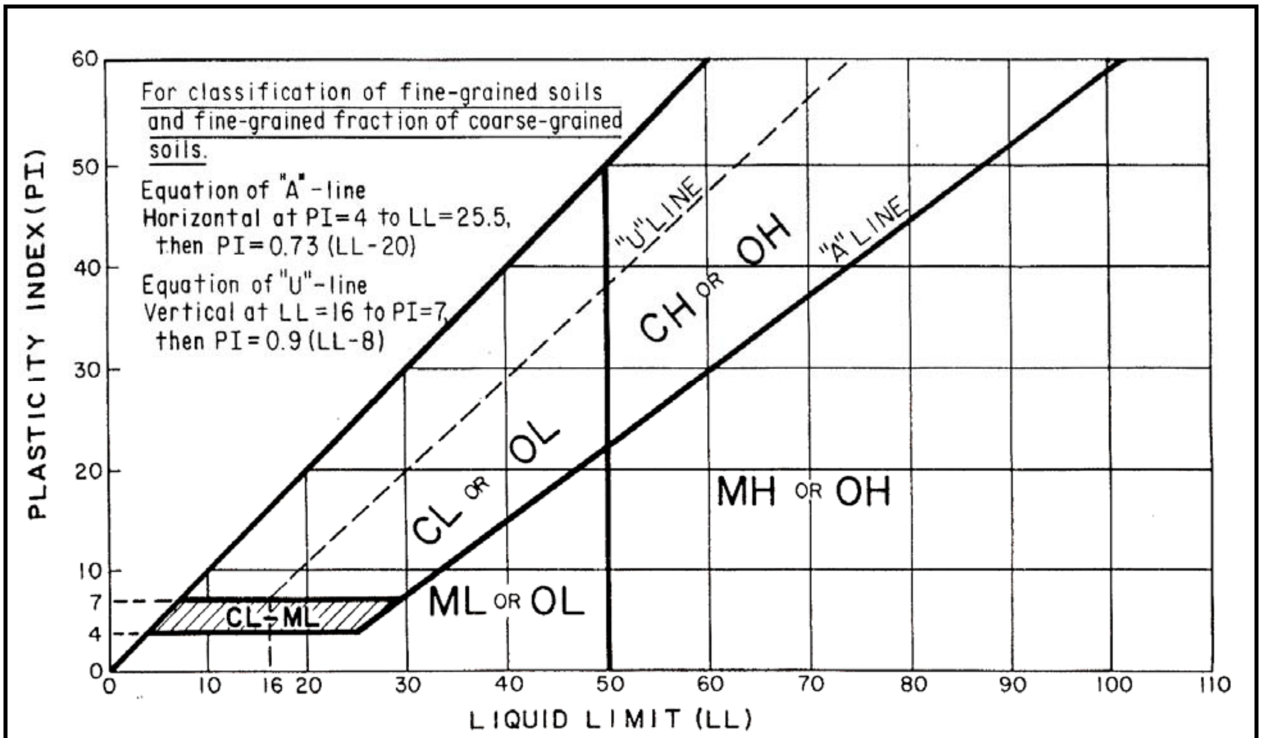

Figure 6-1, Plasticity Chart

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.

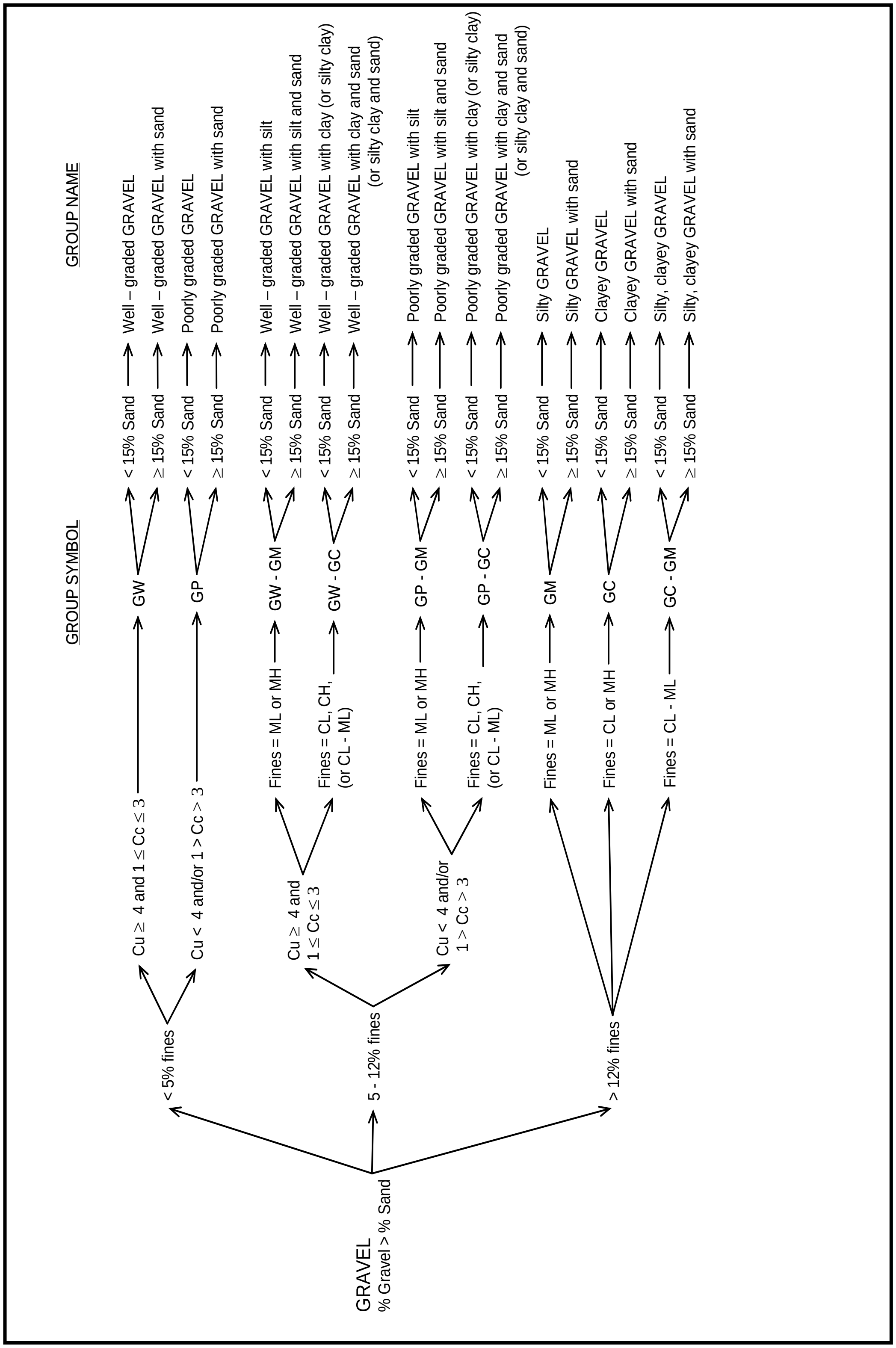

Figure 6-2, Group Symbol and Group Name Coarse-Grained Soils (Gravel) (Subsurface Investigations – Geotechnical Site Characterization – May 2002)

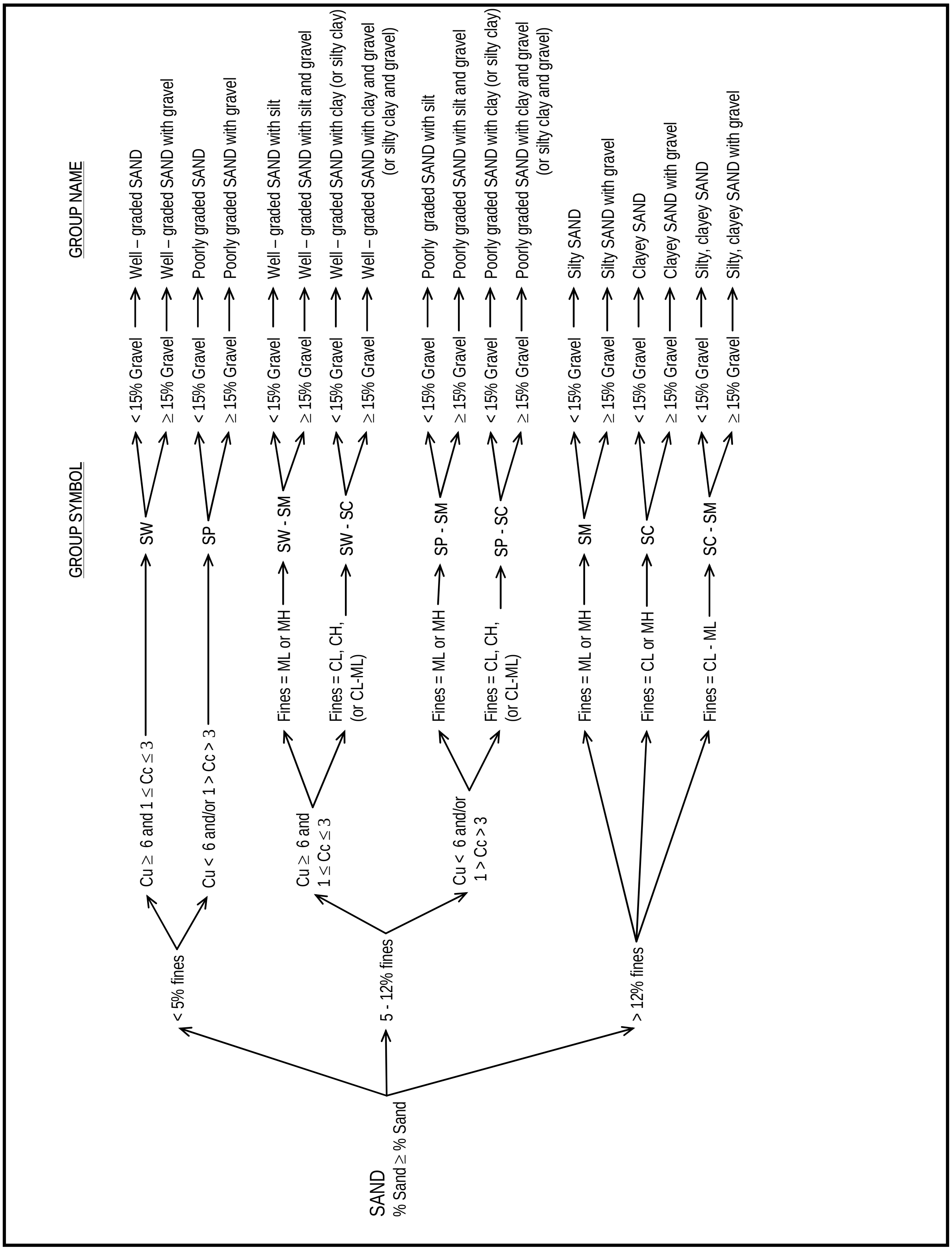

Figure 6-3, Group Symbol and Group Name for Coarse-Grained Soils (Sand) (Subsurface Investigations – Geotechnical Site Characterization – May 2002)

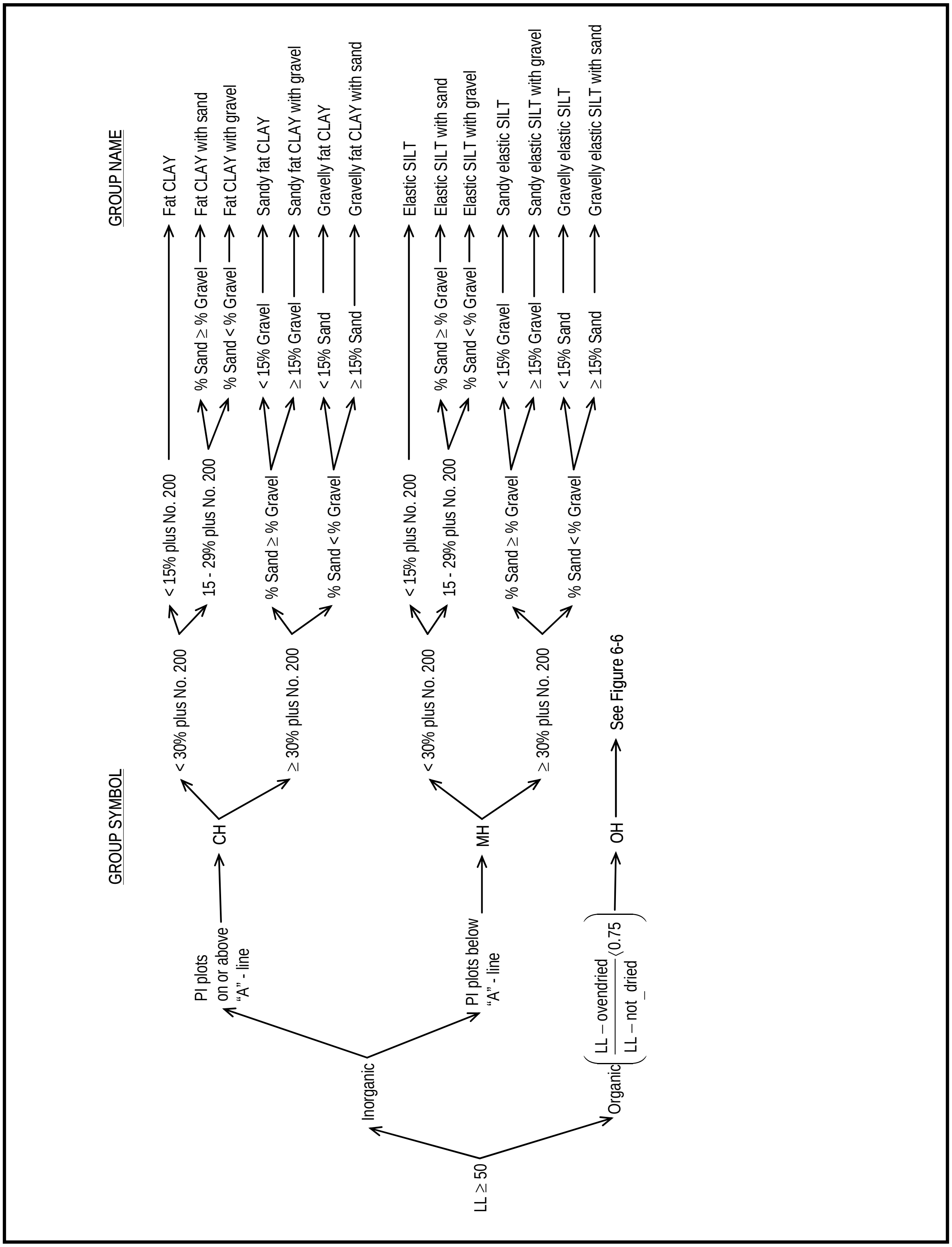

Figure 6-4, Group Symbol and Group Name for Fine-Grained Soils (LL ≥ 50)
(Subsurface Investigations – Geotechnical Site Characterization – May 2002)

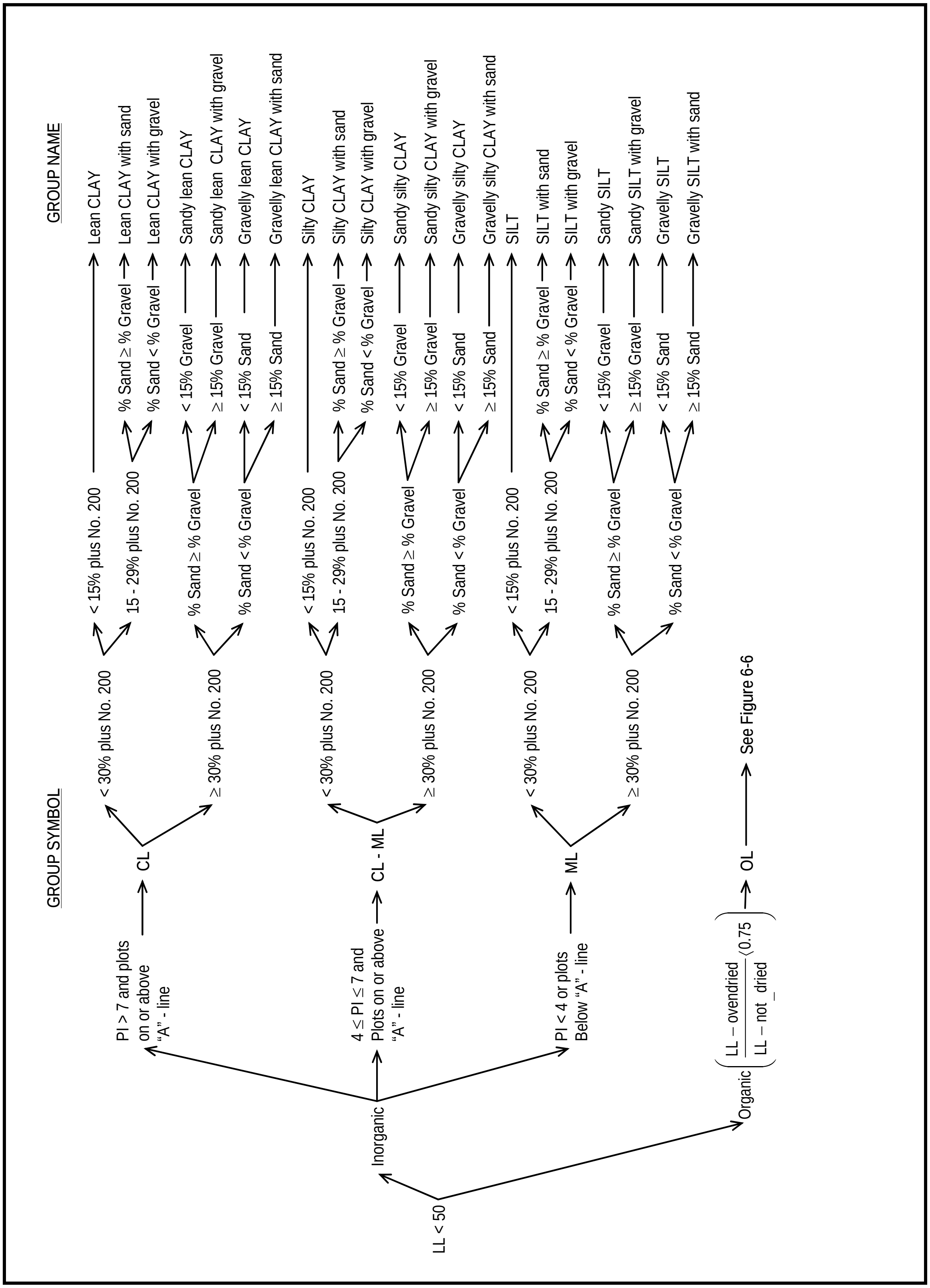

Figure 6-5, Group Symbol and Group Name for Fine-Grained Soils (LL < 50) (Subsurface Investigations – Geotechnical Site Characterization – May 2002)

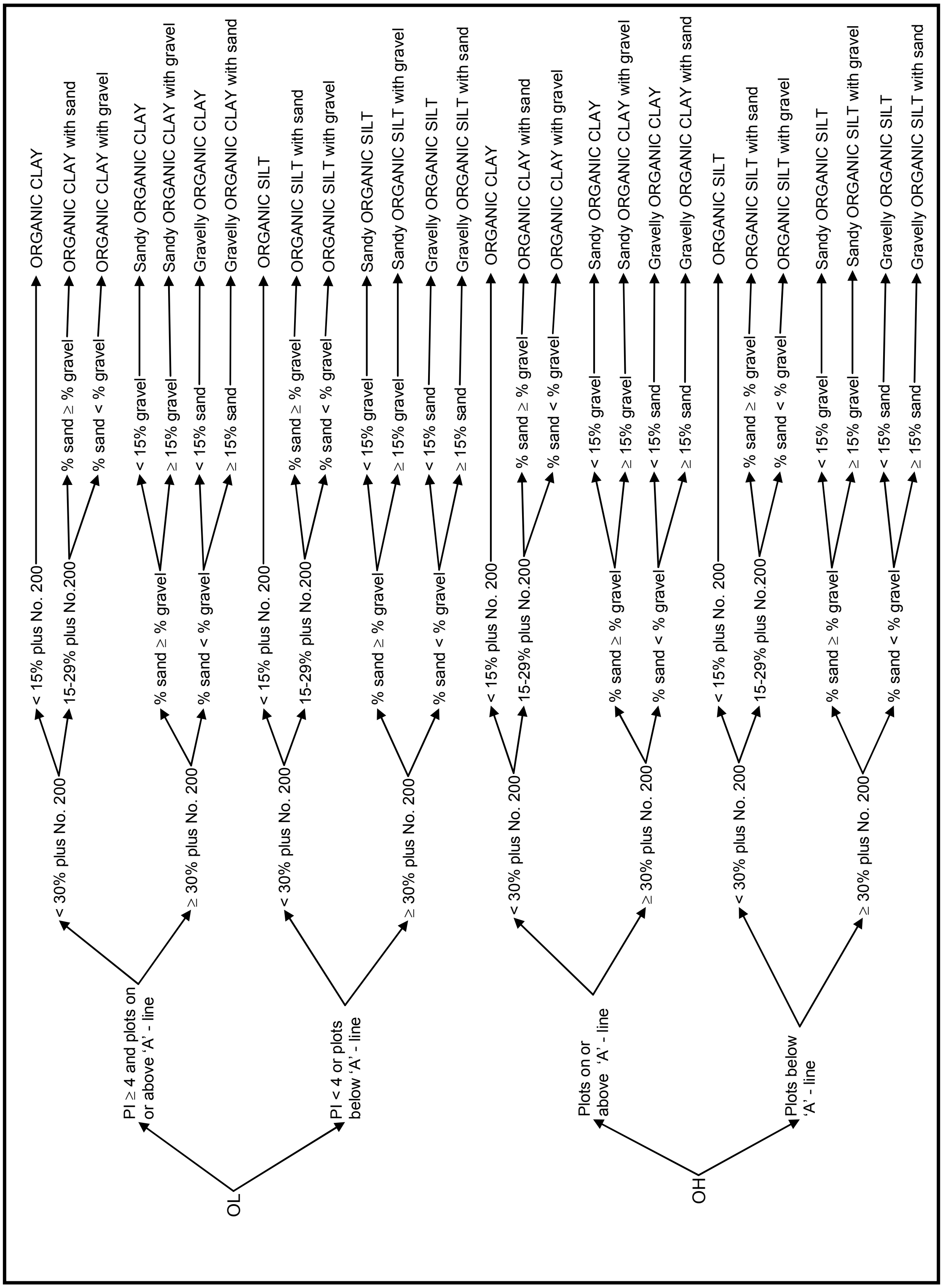

Figure 6-6, Group Symbol and Group Name for Organic Soils (Subsurface Investigations – Geotechnical Site Characterization – May 2002)

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:

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

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

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

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

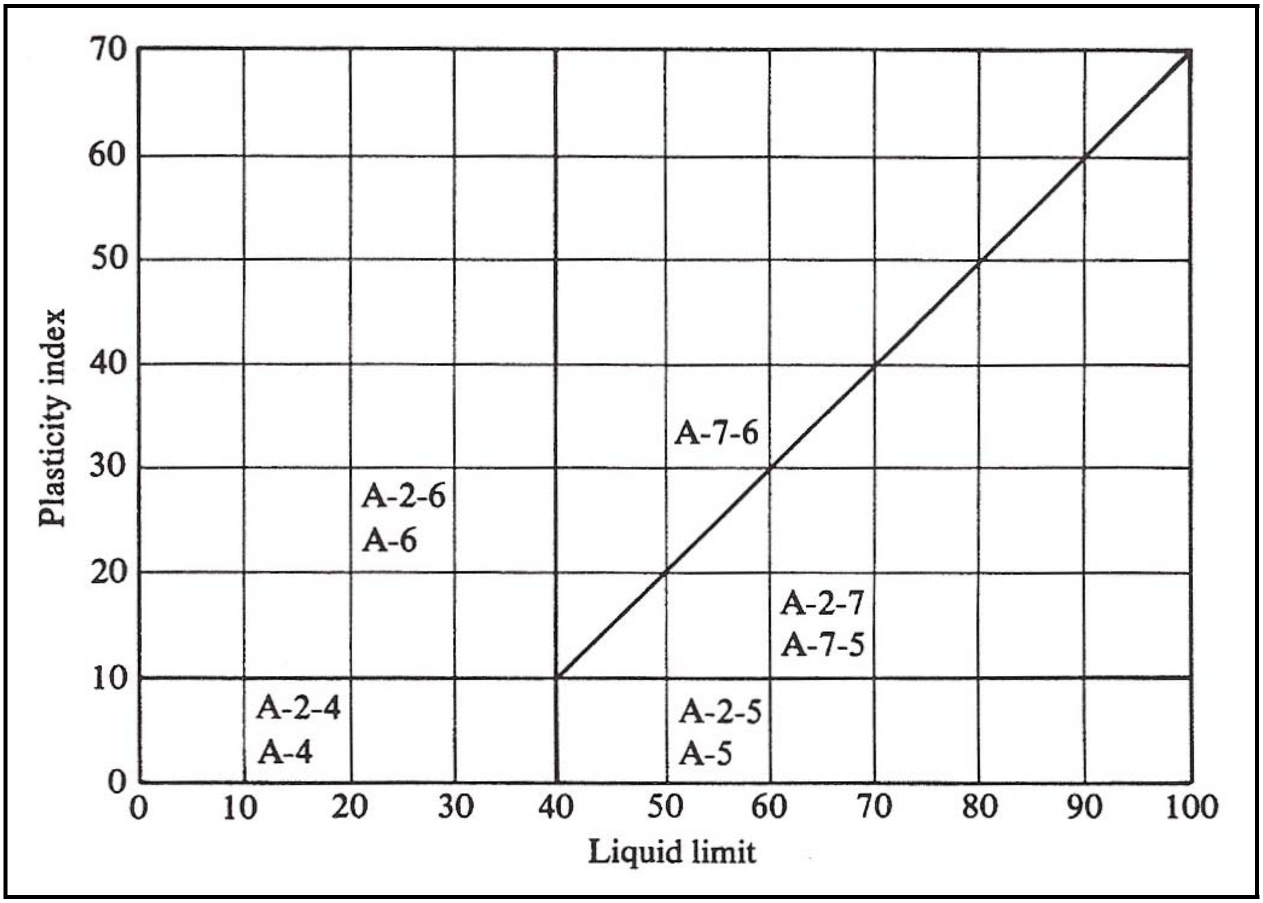

Figure 6-7, Range of LL and PI for Soils in Groups A-2 through A-7
(Subsurface Investigations – Geotechnical Site Characterization – May 2002)

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

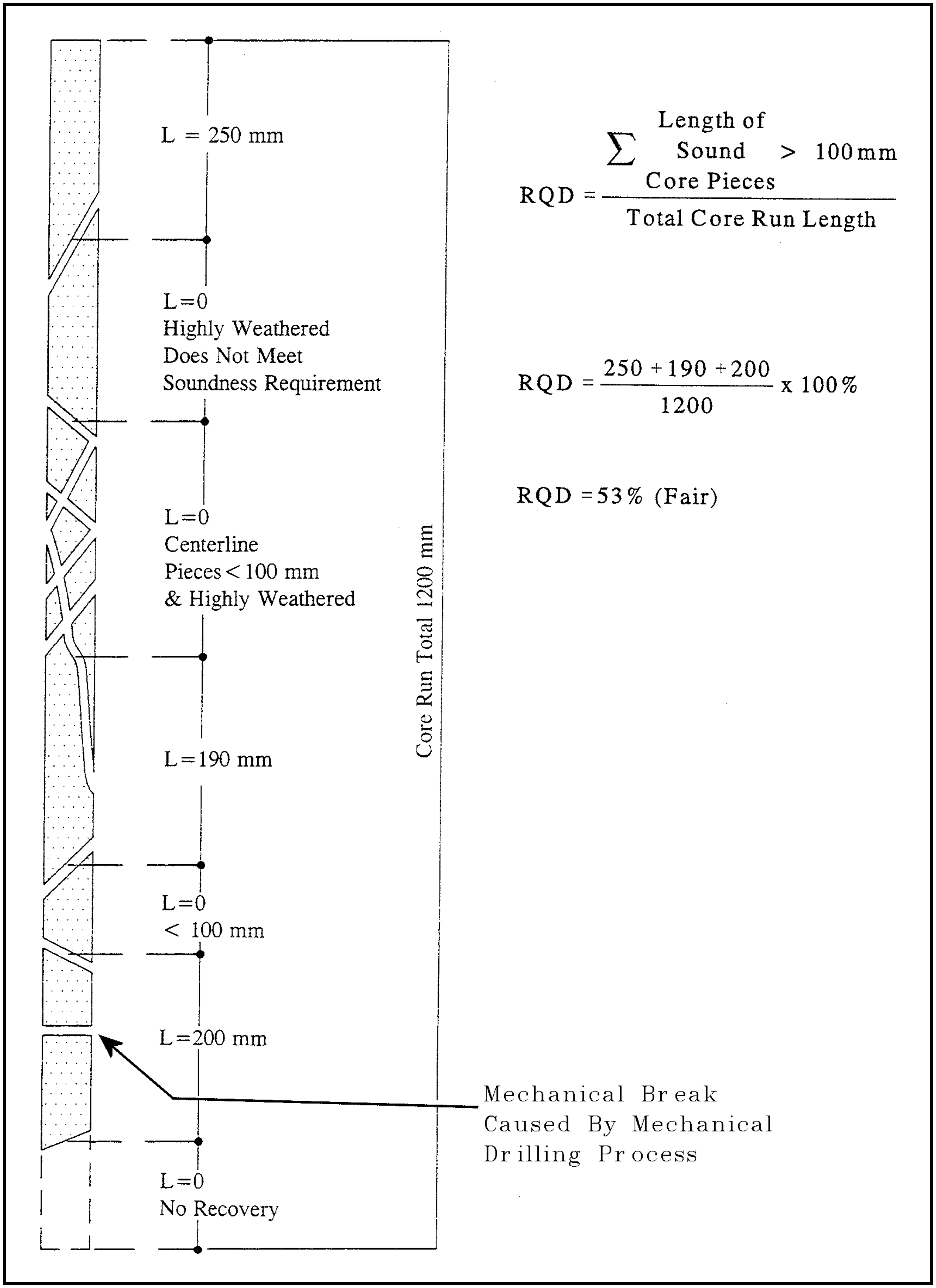

Figure 6-9, RQD Determination
(Subsurface Investigations – Geotechnical Site Characterization – May 2002)

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

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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.

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Chapter 7
GEOMECHANICS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

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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.

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

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

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

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

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

7.8.2 CPT Corrections

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.

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

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

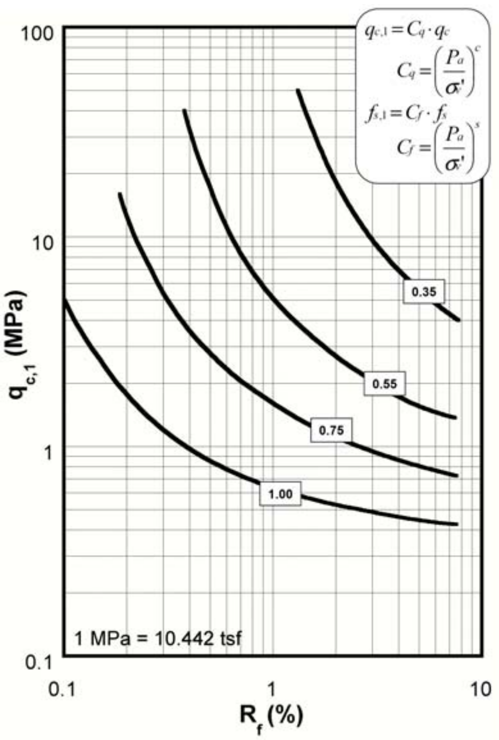
Where,

- 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.

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

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.

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

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 cm^2 cone (diameter, d=35.7mm).

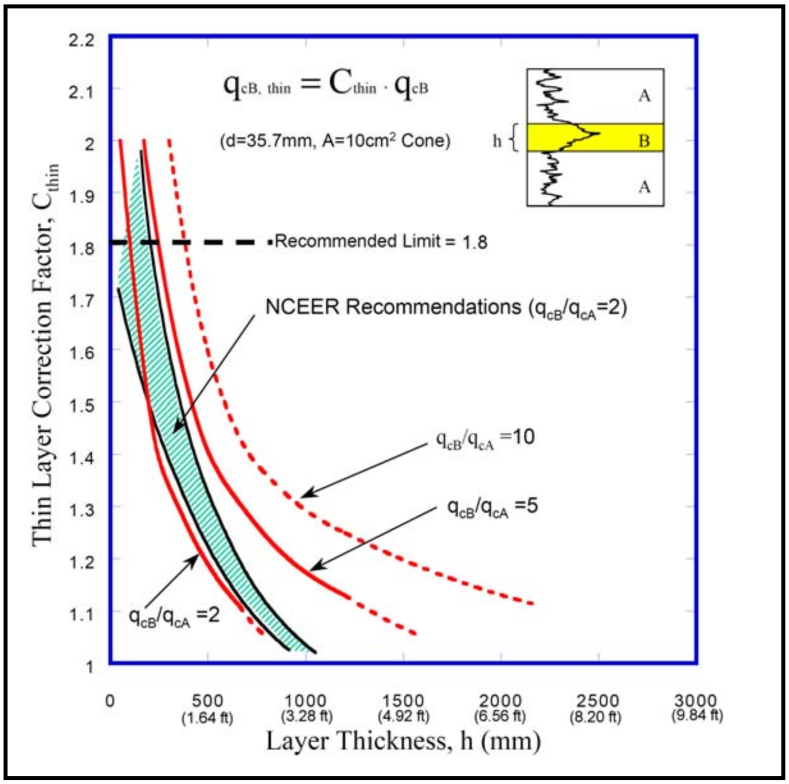


Figure 7-2, CPT Thin Layer Correction (C_{Thin}) (Moss et al., 2006)

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

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

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.

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

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

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.

$$\mathbf{Q_T} = \frac{\mathbf{q_{c,1}} - \sigma_{\mathbf{v}}}{\sigma'_{\mathbf{v}}} \qquad \text{Equation 7-18}$$

$$\mathbf{F_R} = \frac{\mathbf{f_{s,1}}}{\left(\mathbf{q_{c,1}} - \sigma_{\mathbf{v}}\right)} \times \mathbf{100} \qquad \text{Equation 7-19}$$

$$\mathbf{B_q} = \frac{\left(\mathbf{U_2} - \mathbf{U_0}\right)}{\left(\mathbf{q_t} - \sigma_{\mathbf{v}}\right)} \qquad \text{Equation 7-20}$$

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.

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

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.

$$\boldsymbol{t}_{99} = 4\frac{\boldsymbol{D}^2}{\boldsymbol{C_v}}$$ 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.

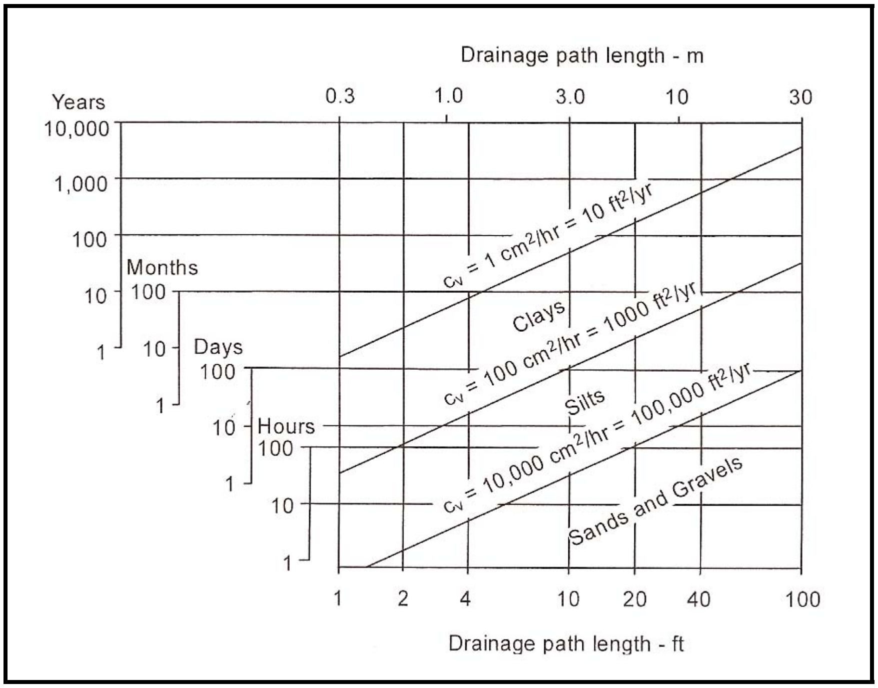


Figure 7-3, Drainage Time Required (Duncan and Wright, 2005)

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.

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

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.

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

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

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

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

- **<u>Underconsolidated (UC; *OCR* < 1):</u>** 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 ($\delta_v = 0$) 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.

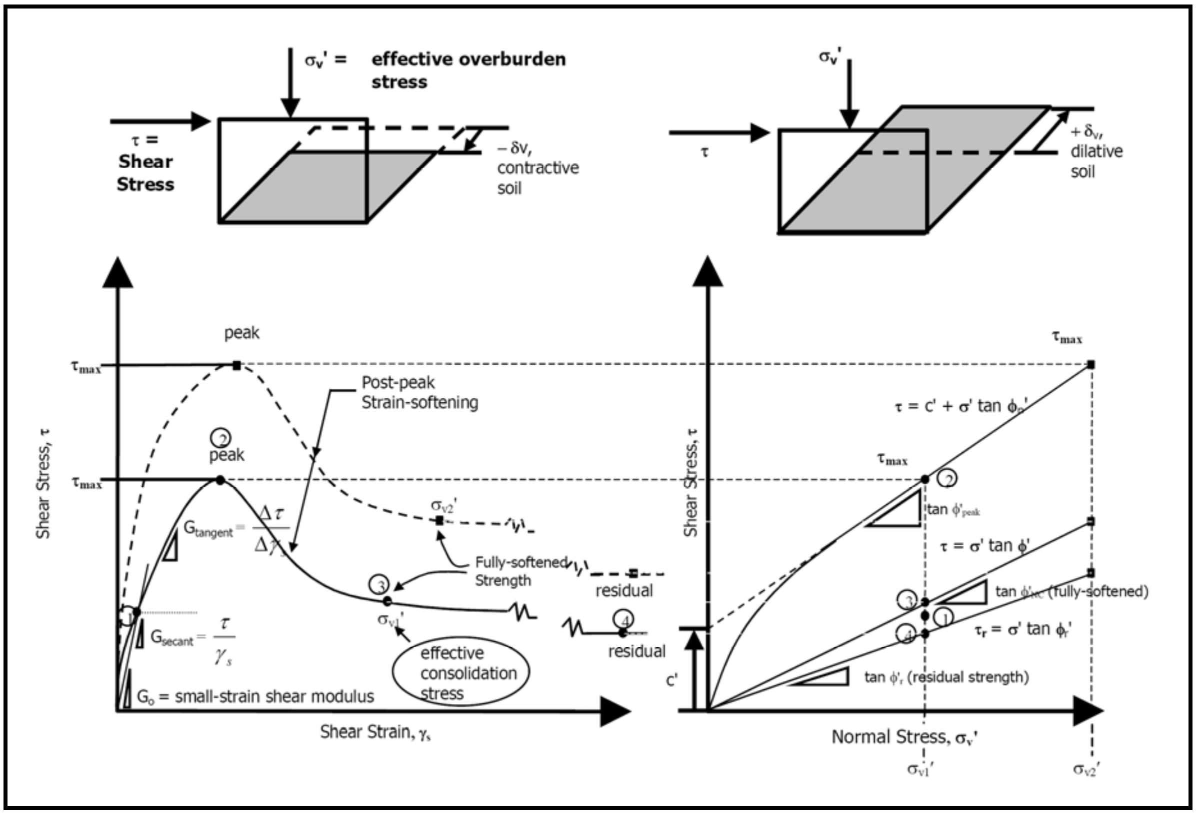


Figure 7-4, Drained Stress-Strain Behavior (Sabatini et al., 2002)

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).

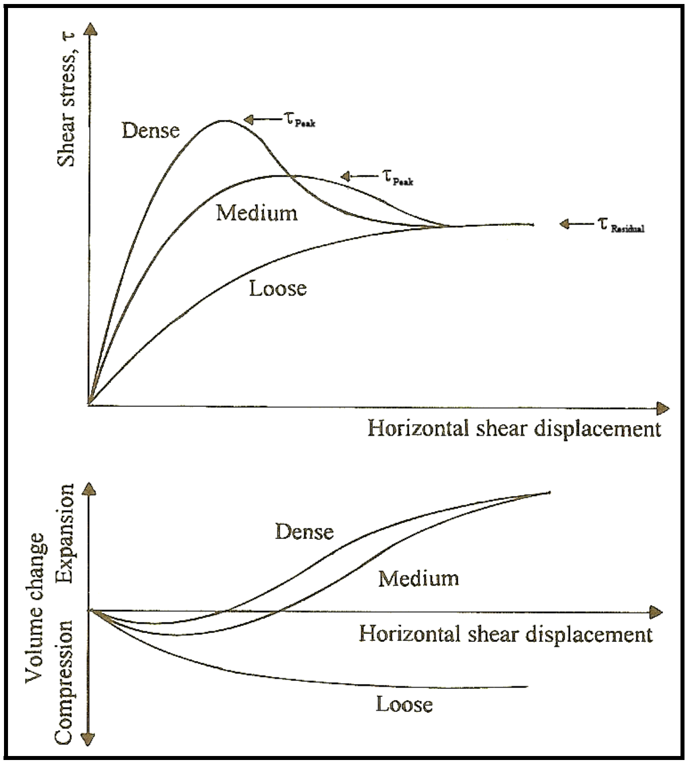


Figure 7-5, Shear Strength Sands (Direct Shear-Test) (Das, 1997)

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.

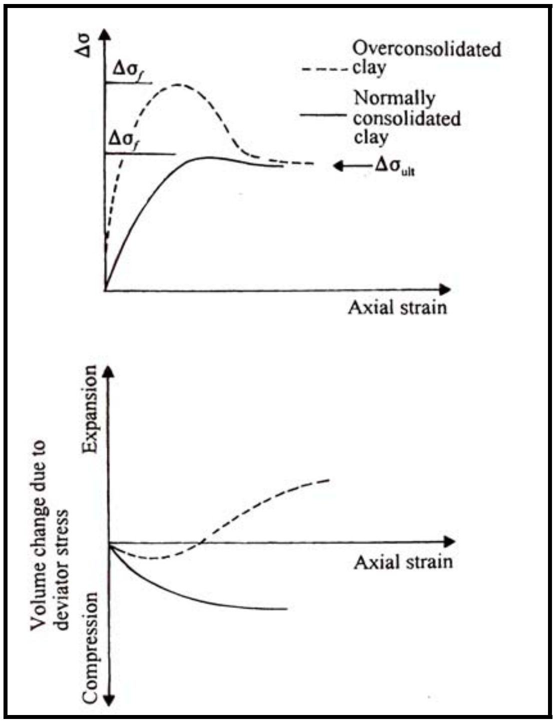


Figure 7-6, Shear Strength of Clay Consolidated Drained Triaxial
(Das, 1997)

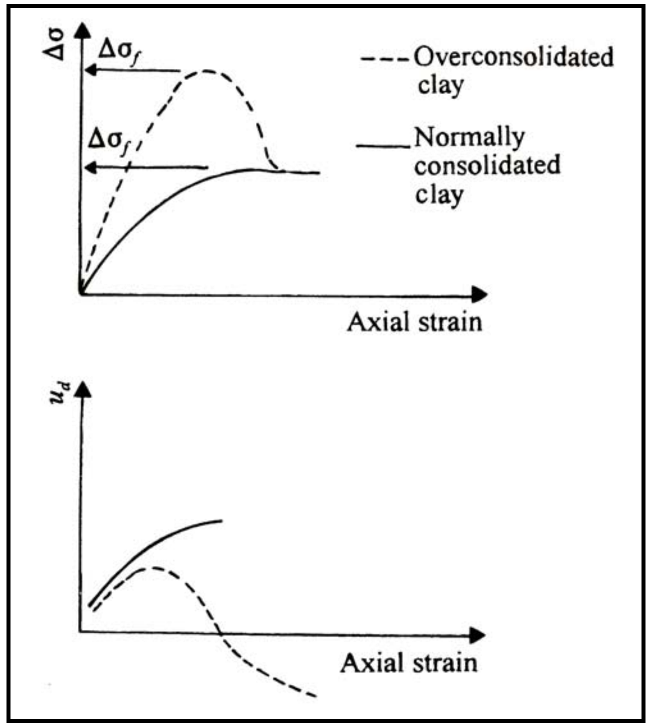


Figure 7-7, Shear Strength of Clay Consolidated Undrained Triaxial
(Das, 1997)

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	√	√	---	---	√	√	---	√
	Sliding Frictional Resistance	√	√	---	---	√	√	---	√
	Sliding Passive Resistance	√	√	---	---	√	√	---	√
	Structural Capacity	√	√	---	---	√	√	---	√
	Lateral Load Analysis (Lateral Displacements)	---	---	√	√	√	√	---	√
	Settlement	---	---	∇	∇	∇	∇	∇	∇
	Global Stability	---	---	√	√	√	√	---	√
Embankment Design	Soil Bearing Resistance	√	√	---	---	√	√	---	√
	Lateral Spread	√	√	---	---	√	√	---	√
	Lateral Squeeze	√	√	---	---	√	√	---	√
	Lateral Displacements	---	---	√	√	√	√	---	√
	Vertical Settlement	---	---	∇	∇	∇	∇	∇	∇
	Global Stability	---	---	√	√	√	√	---	√

[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)

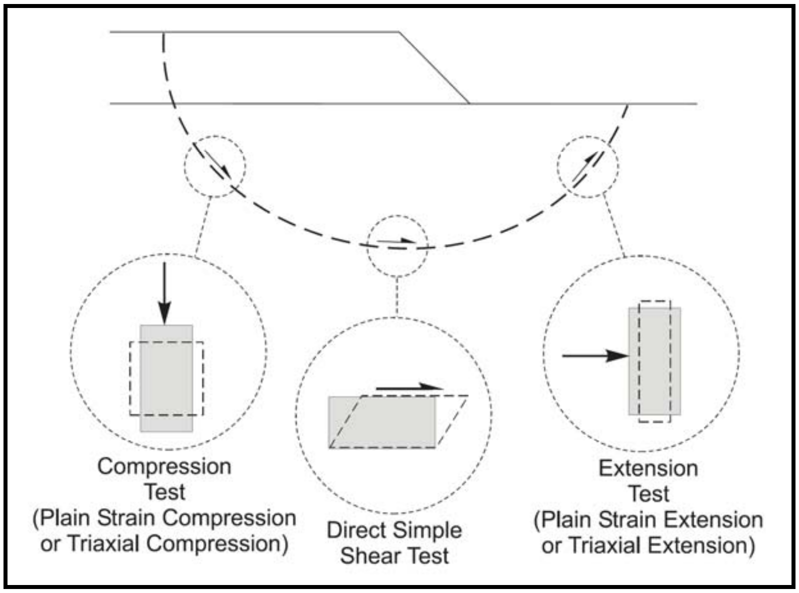


Figure 7-8, Shear Modes for Embankment Stability Shear Failure Surface (Sabatini, 2005)

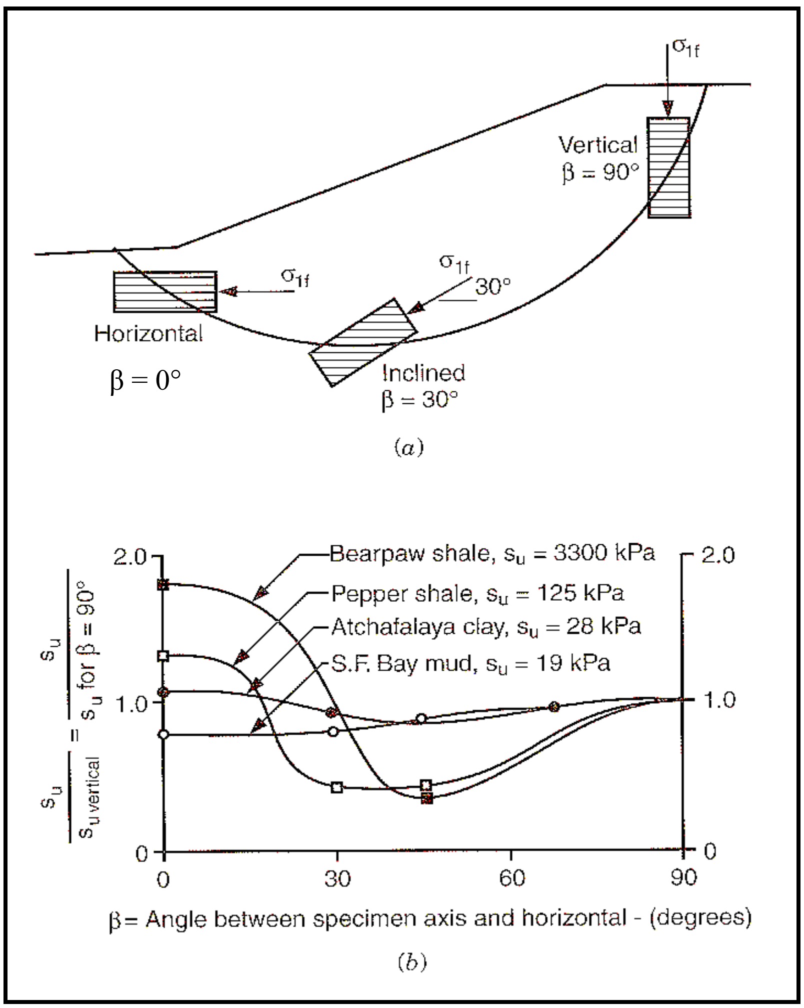


Figure 7-9, τ of Clays and Shales as Function of Failure Orientation (modified from Duncan and Wright, 2005)

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.

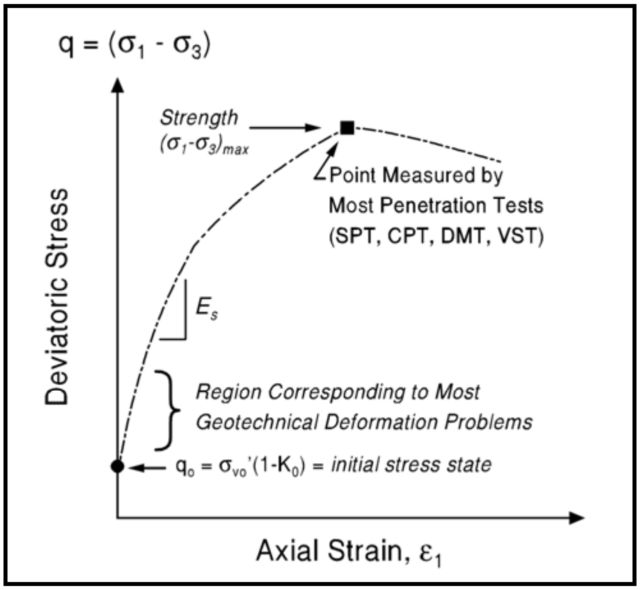


Figure 7-10, Shear Strength Measured by In-Situ Testing (Sabatini, 2005)

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:

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

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).

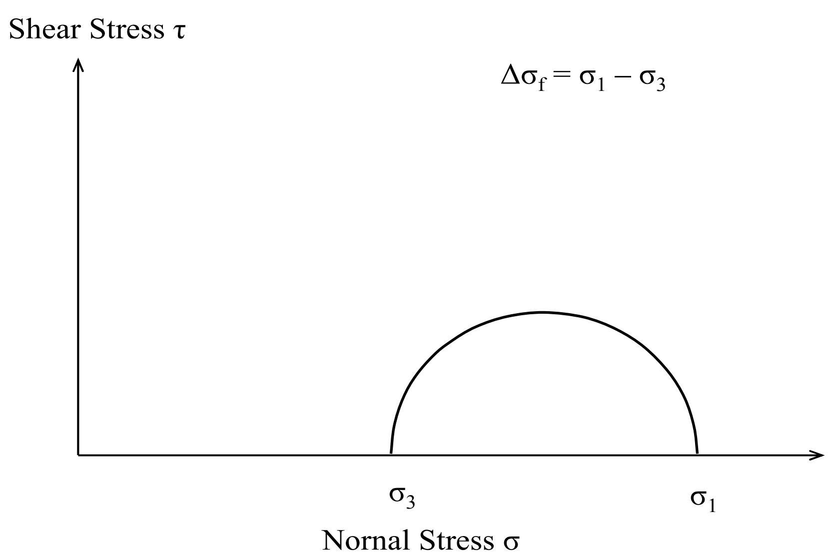


Figure 7-11, Total Principal Stresses

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

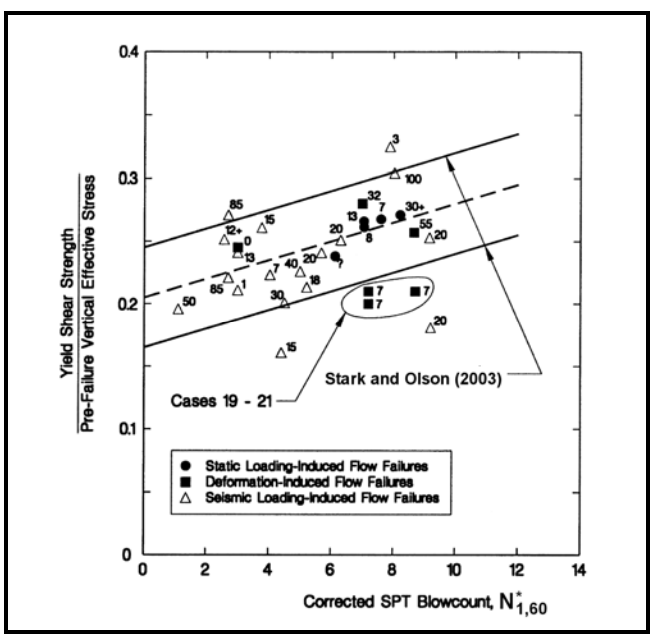


Figure 7-12, Yield Shear Strength Ratio - SPT Blowcount Relationship (Olson, 2001, Olson and Stark, 2003)

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.

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

Where,

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

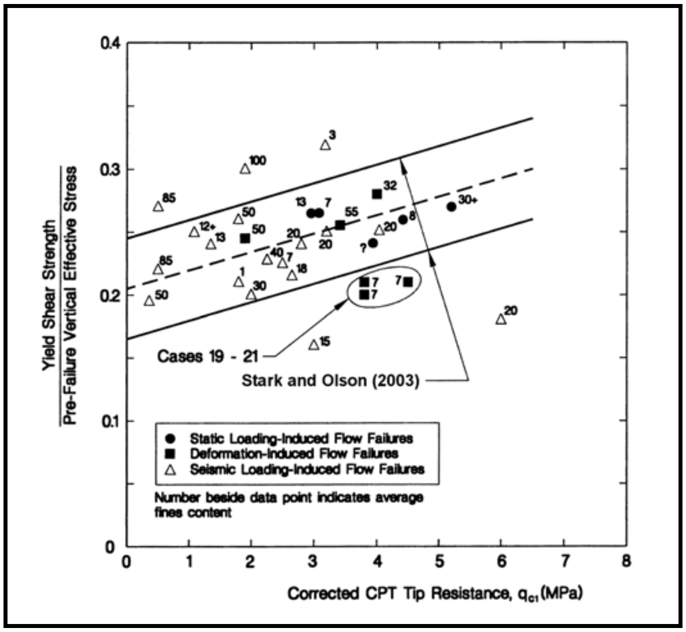


Figure 7-13, Yield Shear Strength Ratio - CPT Tip Resistance Relationship (Olson, 2001, Olson and Stark, 2003)

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 (ϕ = 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

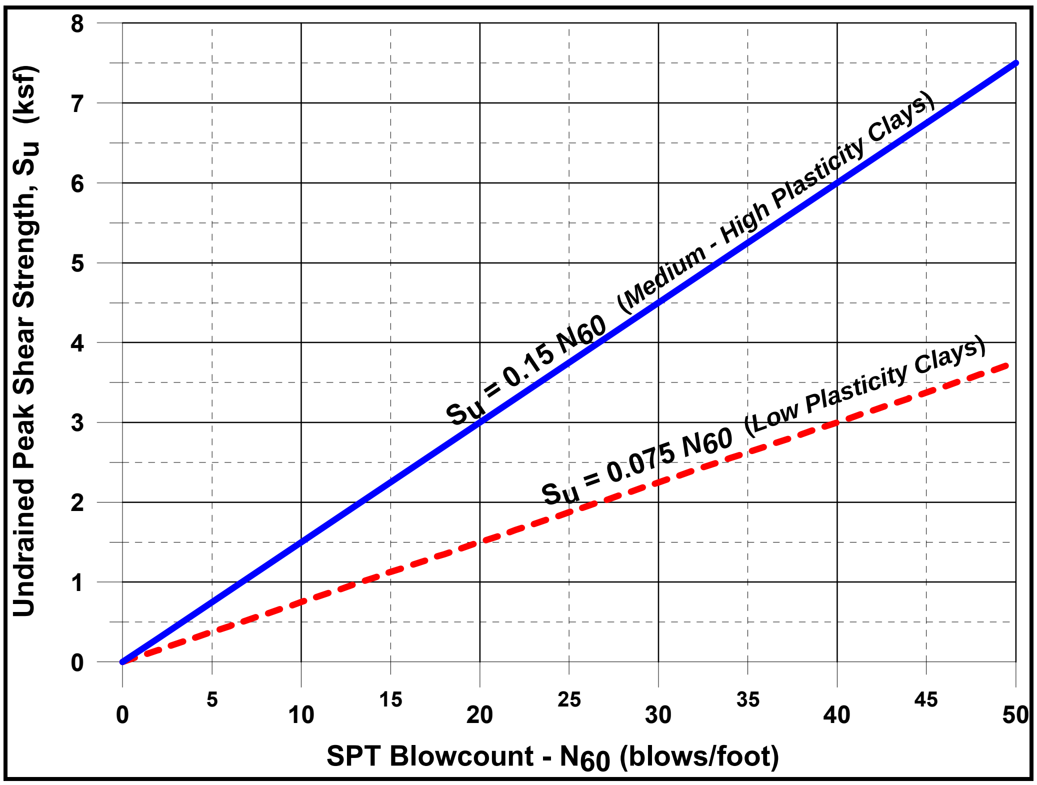


Figure 7-14, Undrained Shear Strength – SPT Relationship (McGregor and Duncan, 1986)

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.

Equation 7-32

$$\tau = \mathbf{c} = \frac{\mathbf{q_c} - \sigma_{\mathbf{vo}}}{\mathbf{N_k^*}}$$

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.

$$\tau = \mu \mathbf{S_{vane}}$$

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.

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

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).

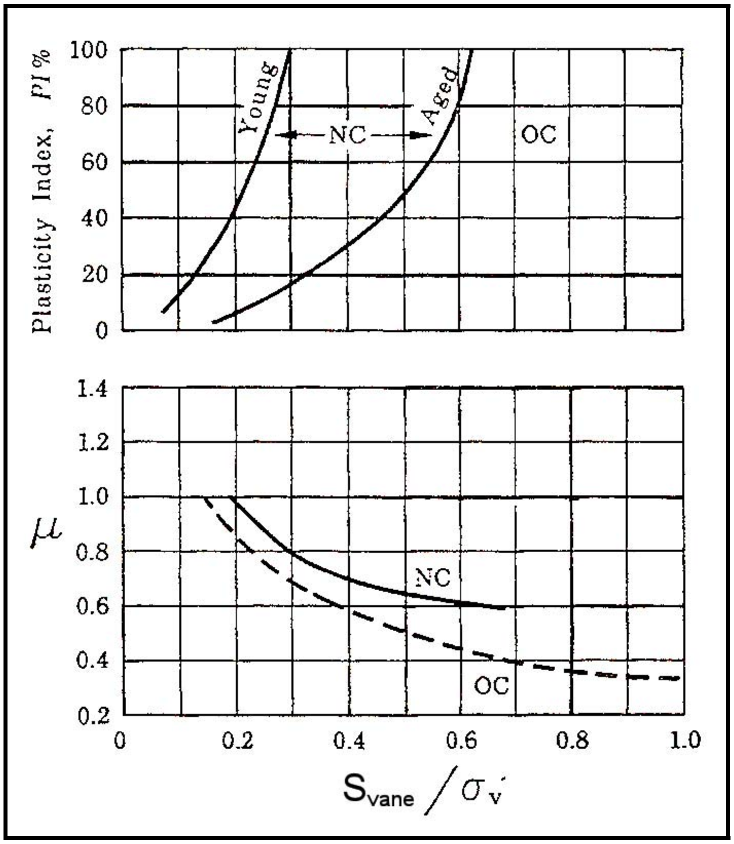


Figure 7-15, Vane Shear Correction Factor (Aas, et. al., 1986)

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.

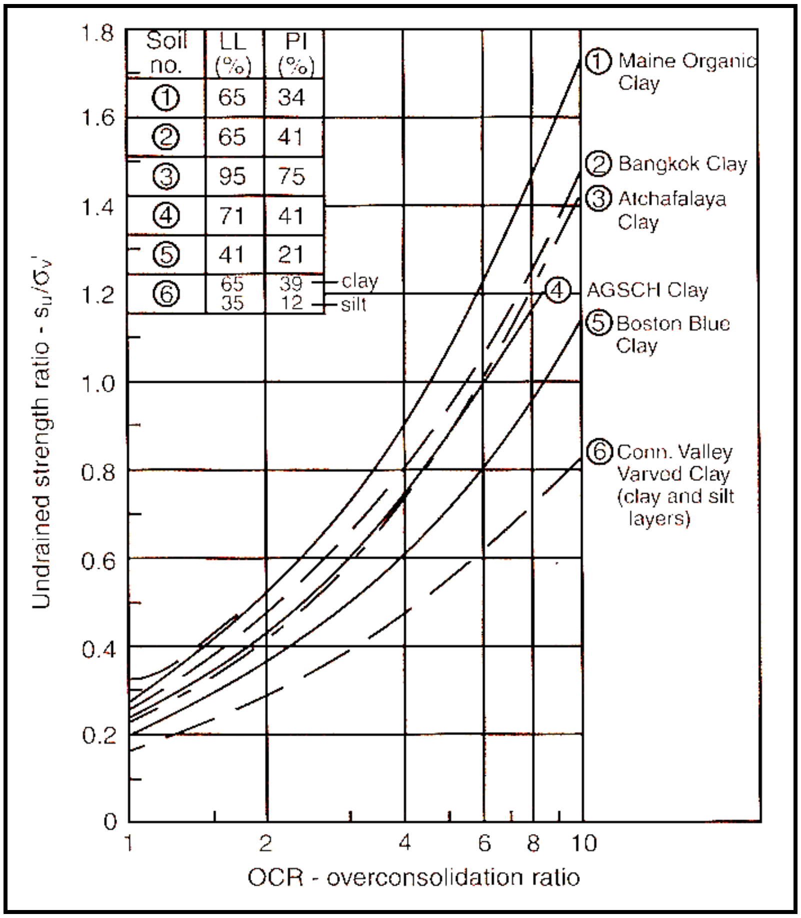


Figure 7-16, Undrained Shear Strength Ratio and OCR Relationship (Ladd et al., 1977)

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.

$$S_t = \frac{\tau}{\tau_R} \qquad \text{Equation 7-36}$$

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.

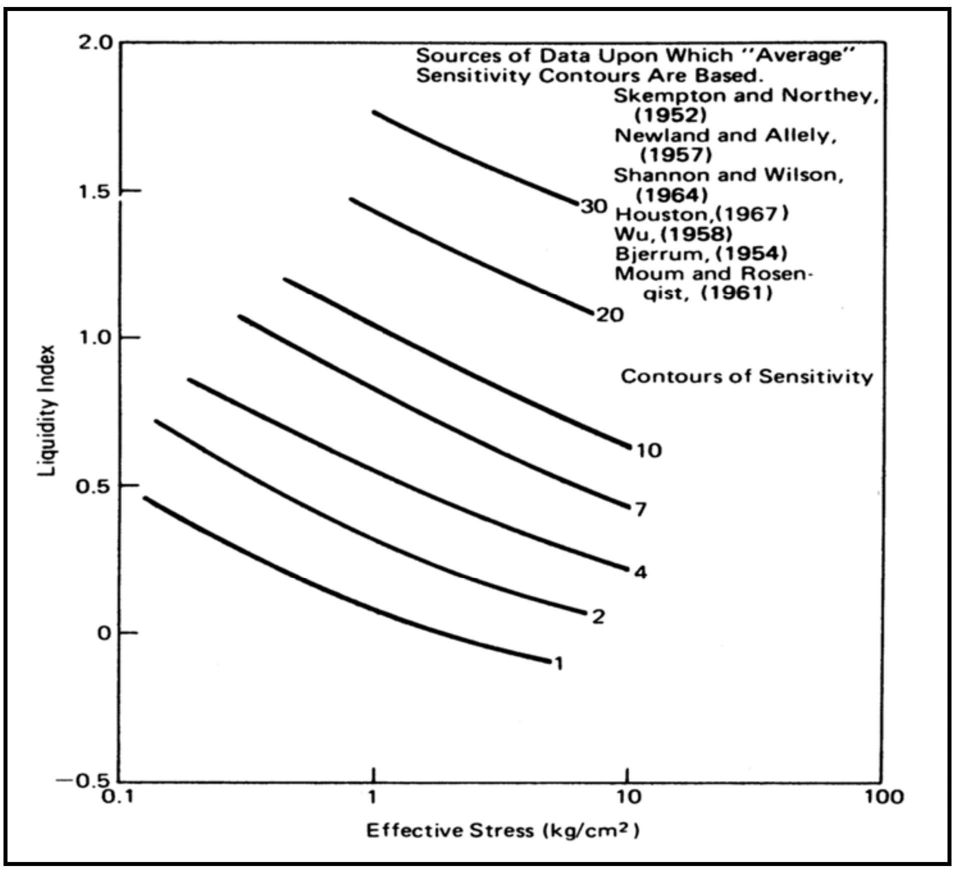


Figure 7-17, Sensitivity based on Liquidity Index and σ'_{vo} (Mitchell, 1993)

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

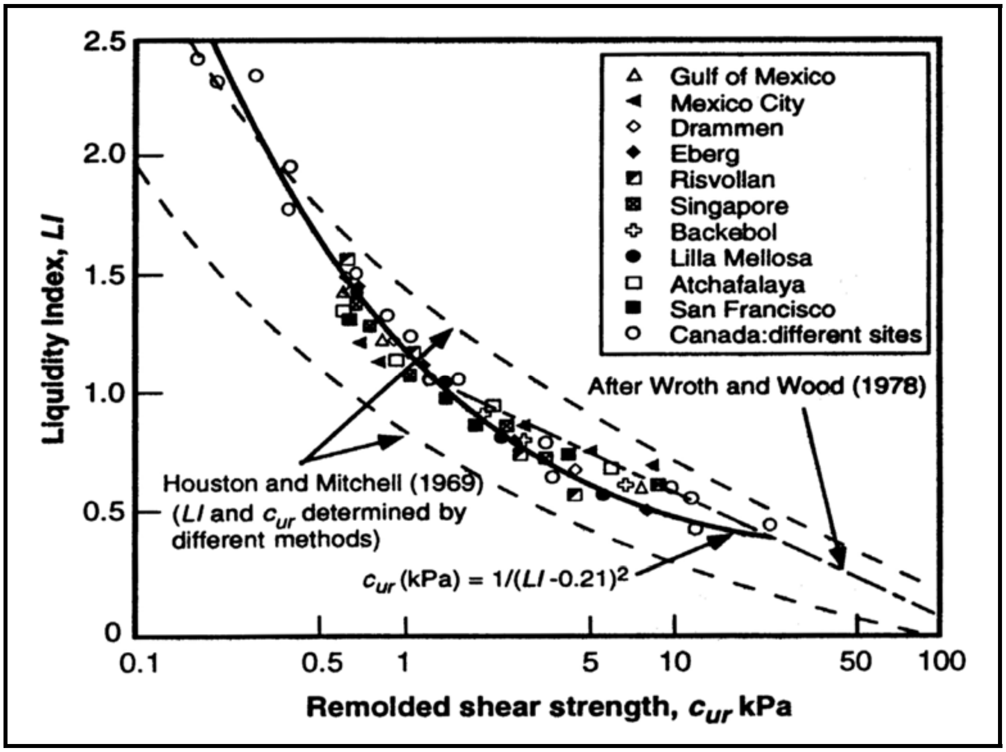


Figure 7-18, Remolded Shear Strength vs Liquidity Index (Mitchell, 1993)

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.

$$\mathbf{LI = \frac{(w - PL)}{(LL - PL)}}$$ 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).

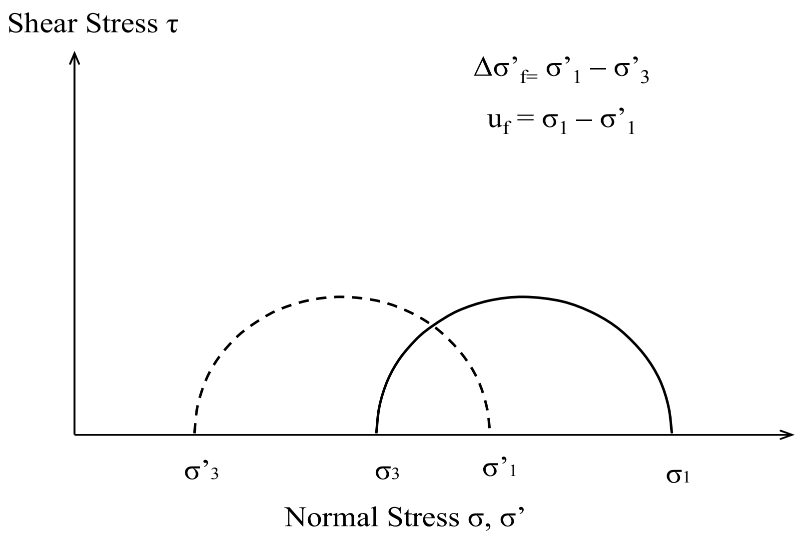


Figure 7-19, Effective Principal Stresses

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

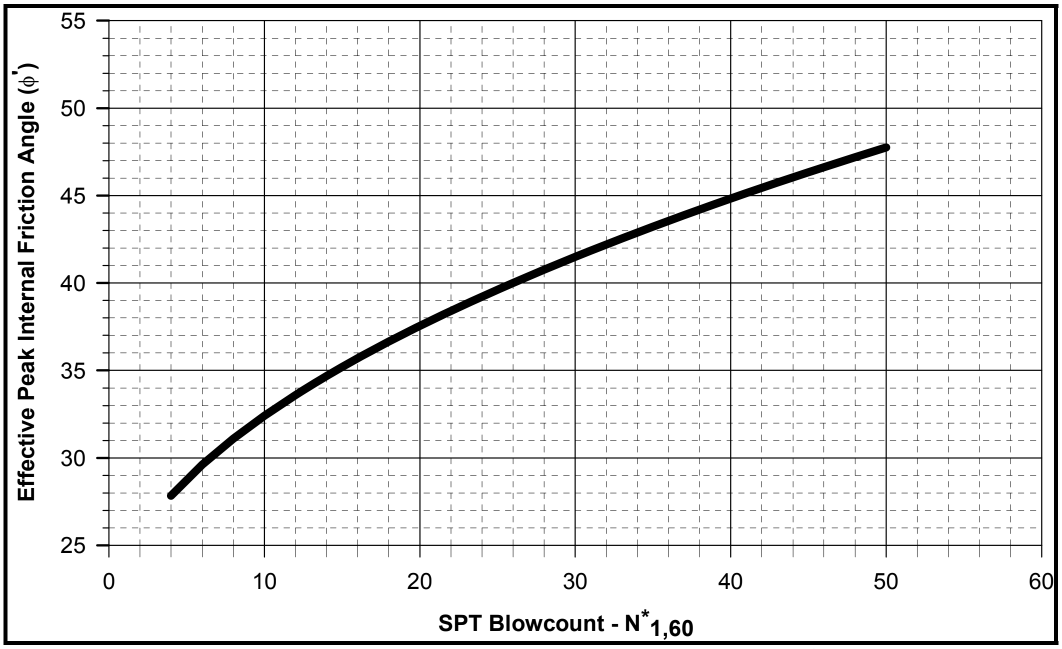


Figure 7-20, Effective Peak Friction Angle and SPT ($N^*_{1,60}$) Relationship (Based on Hatanaka and Uchida, 1996)

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.

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

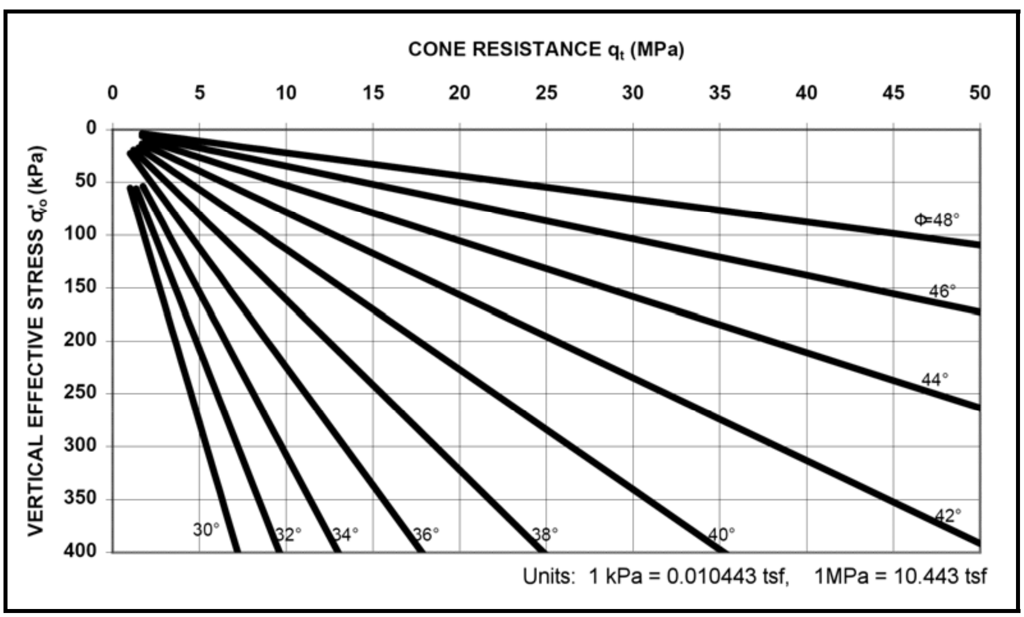


Figure 7-21, Effective Peak Friction Angle and CPT (q_c) Relationship (Robertson and Campanella, 1983)

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

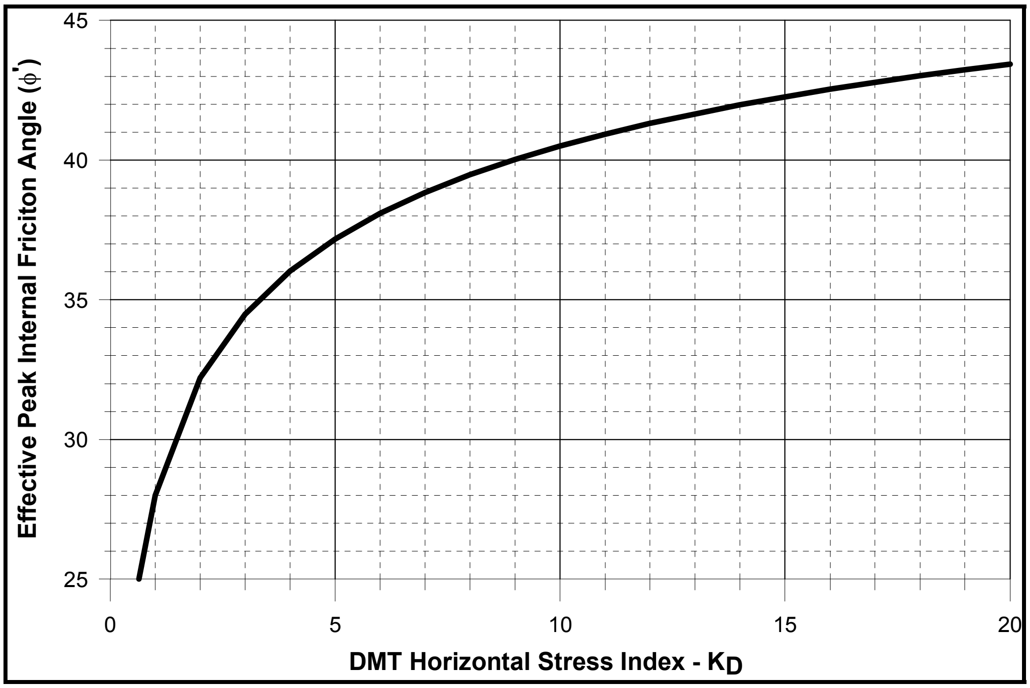


Figure 7-22, Effective Peak Friction Angle and DMT (K_D) Relationship (Robertson and Campanella, 1991)

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).

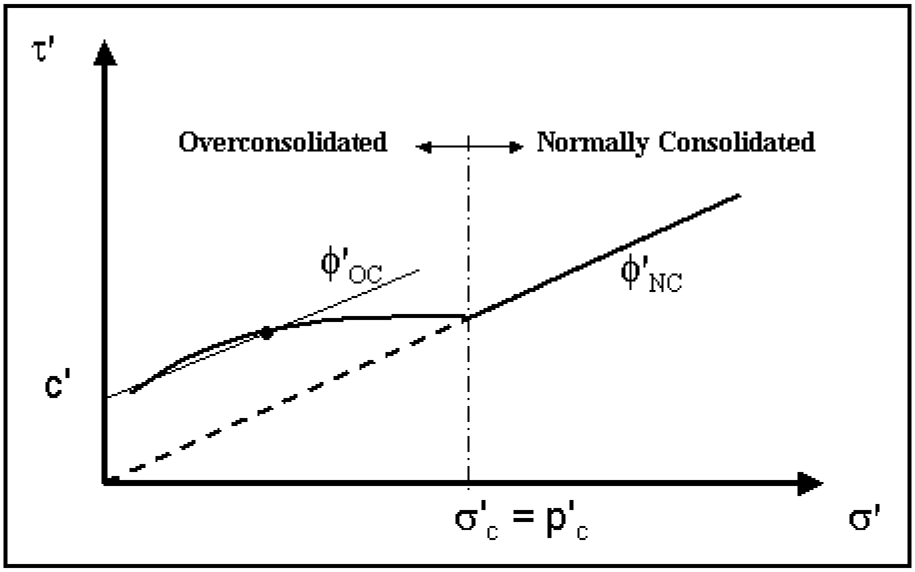


Figure 7-23, Overconsolidated Clay Failure Envelope (CUw/pp Triaxial Test)

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.

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

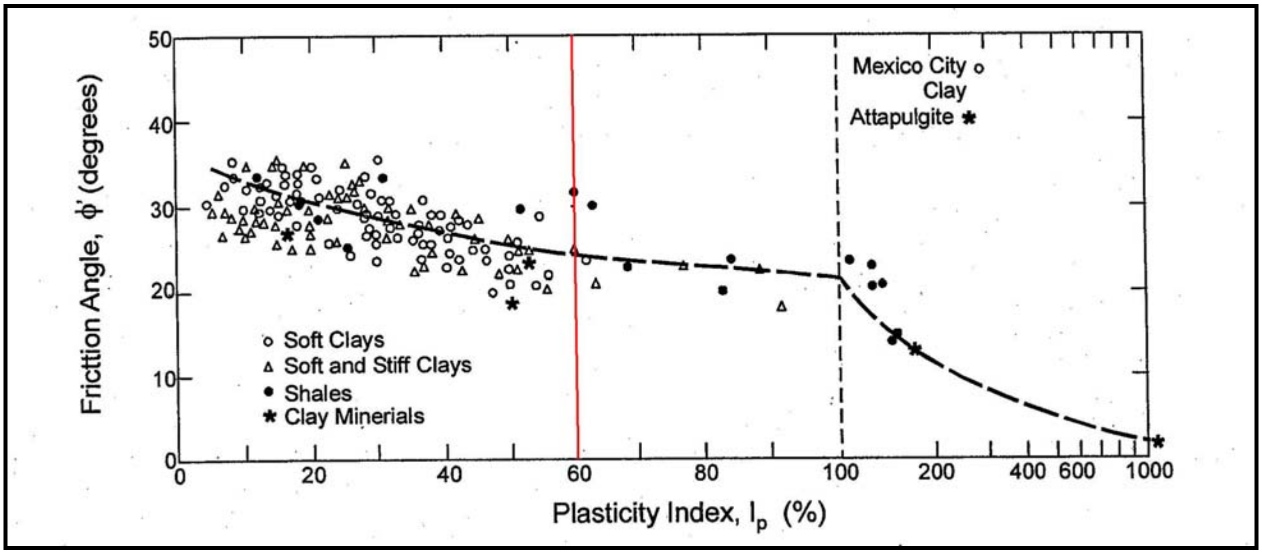


Figure 7-24, Plasticity Index versus Drained Friction Angle For NC Clays (Terzaghi, Peck, and Mesri, 1996)

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).

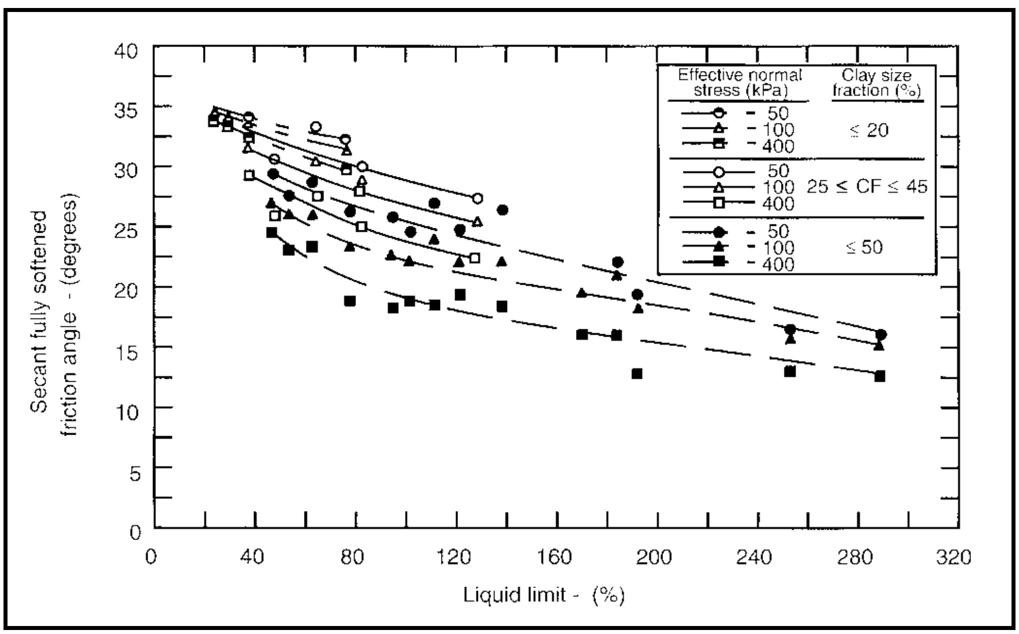


Figure 7-25, Fully Softened (NC) Friction Angle and Liquid Limit Relationship (Stark and Eid, 1997)

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).

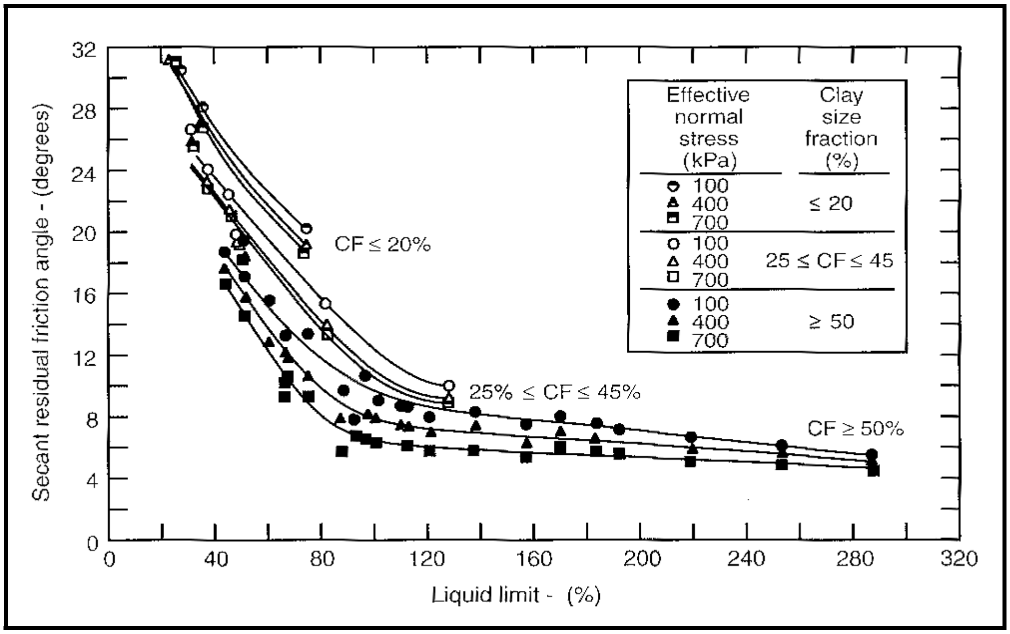


Figure 7-26, Drained Residual Friction Angle and Liquid Limit Relationship (Stark and Eid, 1994)

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:

- **Design-Build Projects:** 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

- **Traditional Design-Bid-Build W/Existing Embankments:** 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.

- **Traditional Design-Bid-Build On New Alignment:** 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 SCDOT Borrow Specifications

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.

- **Group A:** 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.

- **Group B:** 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.

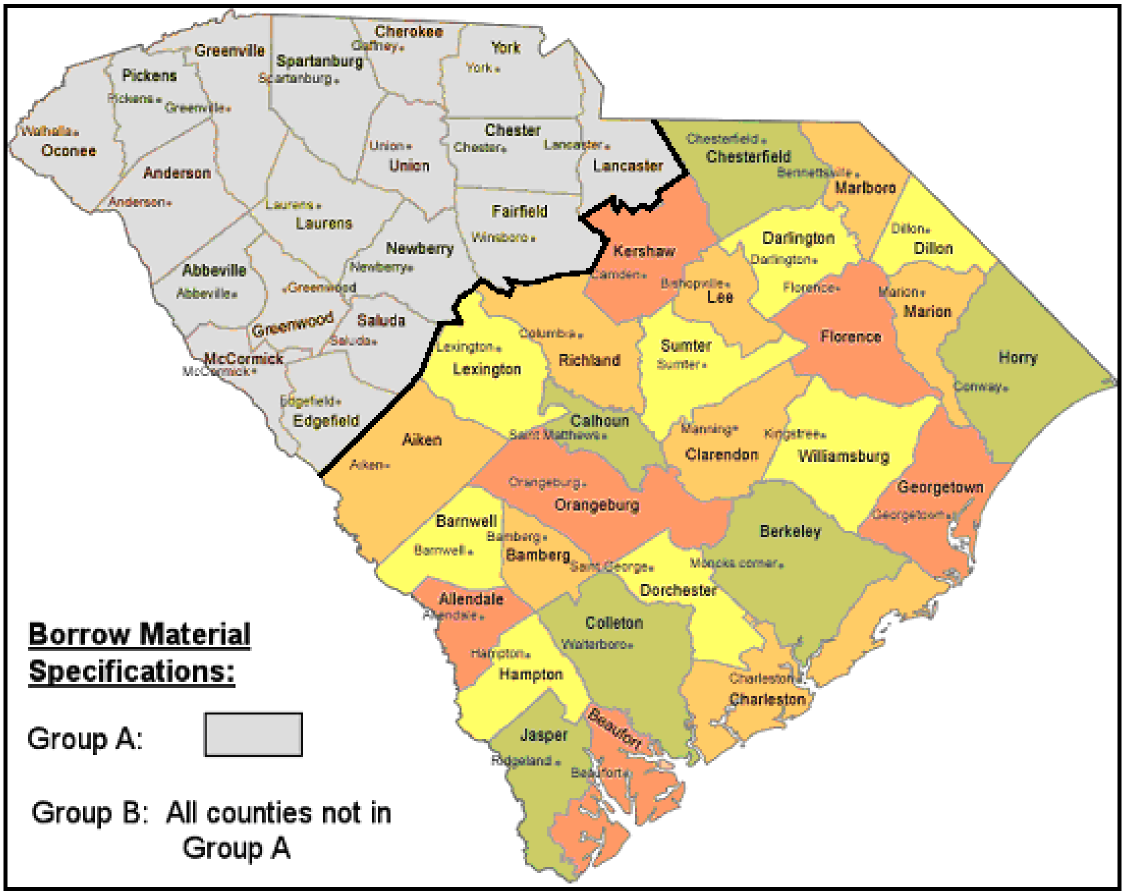


Figure 7-27, Borrow Material Specifications By County

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

- **Group A**: 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.

- **Group B:** 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^o with a $\pm$1 standard deviation range of $29.9^o < \phi' < 40.5^o$. A conservative lower bound of 27.3^o 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 $28^o < \phi' < 32^o$.

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.

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

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

$$\mathbf{C_c = 1.15(e_o - 0.35)}$$ 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.

$$C_{\varepsilon c} = \frac{C_c}{(1 + e_o)}$$ 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.

$$C_{\varepsilon r} = \frac{C_r}{(1 + e_o)}$$ 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.

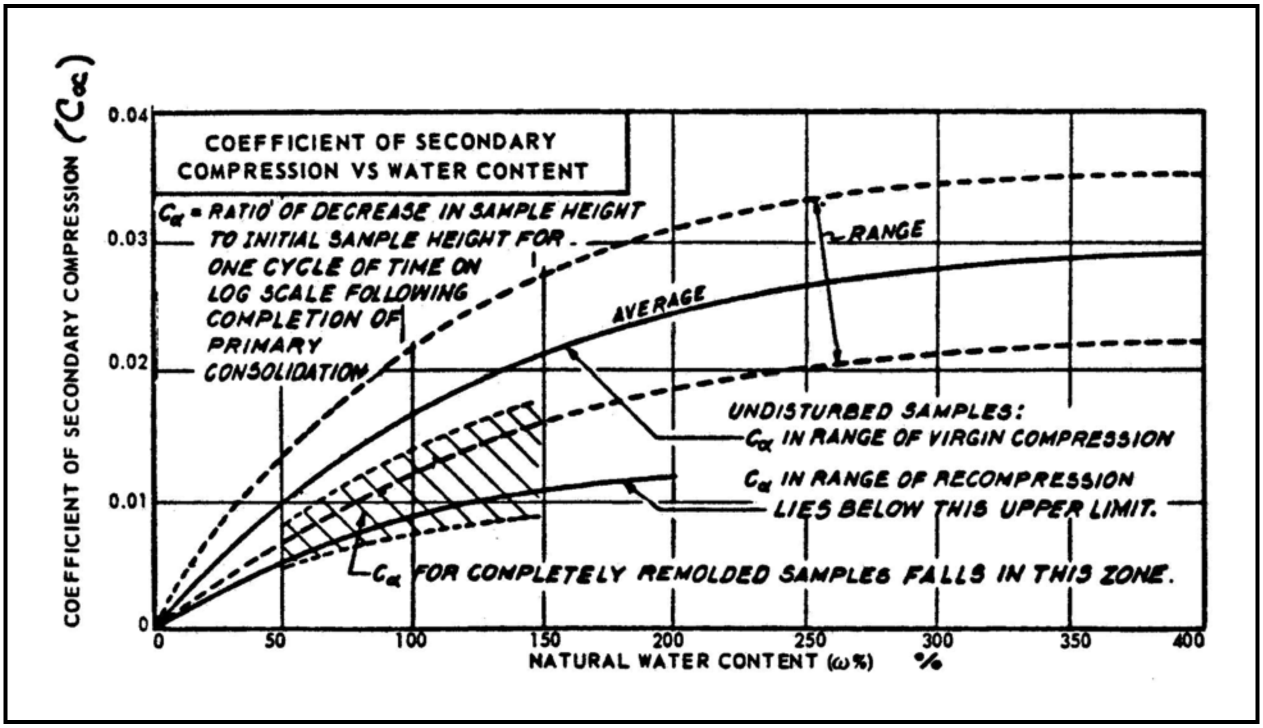


Figure 7-30, Secondary Compression Index Chart
NAVFAC DM-7.1, 1982

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.

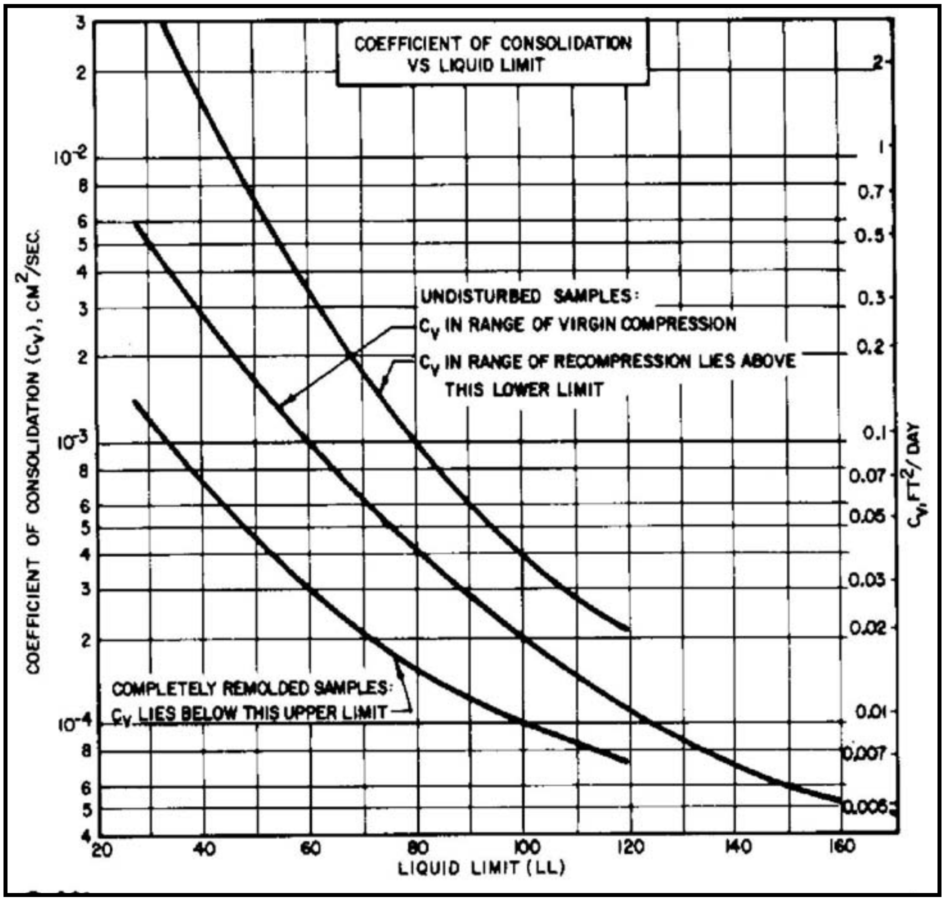


Figure 7-31, Consolidation Coefficient and Liquid Limit Relationship
NAVFAC DM-7.1, 1982

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.

$$\sigma'_p = \frac{c_u}{(0.11 + 0.0037PI)}$$ 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	A	B	C	D	E
		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				
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

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Chapter 8

GEOTECHNICAL

LRFD DESIGN

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

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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:

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

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

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

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 ≥ 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

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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.

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

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.

$$\frac{R_n}{Q} = \frac{\text{Resisting Force}}{\text{Driving Force}} = FS \geq \frac{1}{\varphi}$$ 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 Driven Piles

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 <u>Drilled Shafts</u>

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

FINAL

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

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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:

- *<u>Construction</u>*: Common types of structures are usually associated with less construction risk due to the familiarity of the construction procedures.
- *<u>Structure Selection</u>*: 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.
- *<u>Design/Construction Methodology</u>*: Misapplication of methodologies in design (i.e. using unaccepted design methods) or construction (i.e. misapplication of ground improvement method).
- *<u>Design Experience</u>*: 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 <u>Embankment Terminology and Deformation Notations</u>

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.

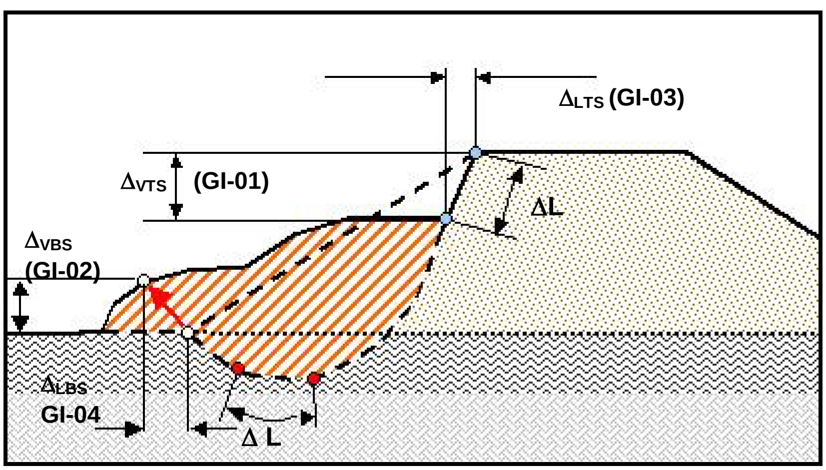

Figure 10-1, Embankment Circular Arc Instability

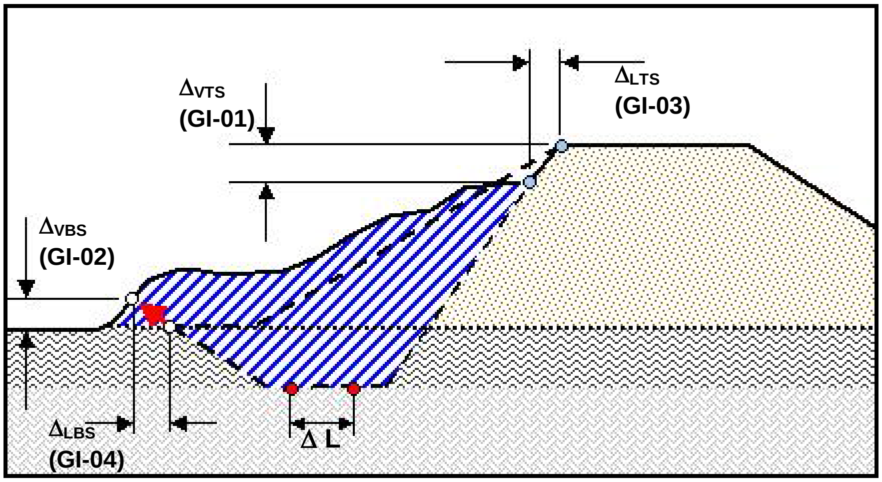

Figure 10-2, Embankment Sliding Block Instability

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.

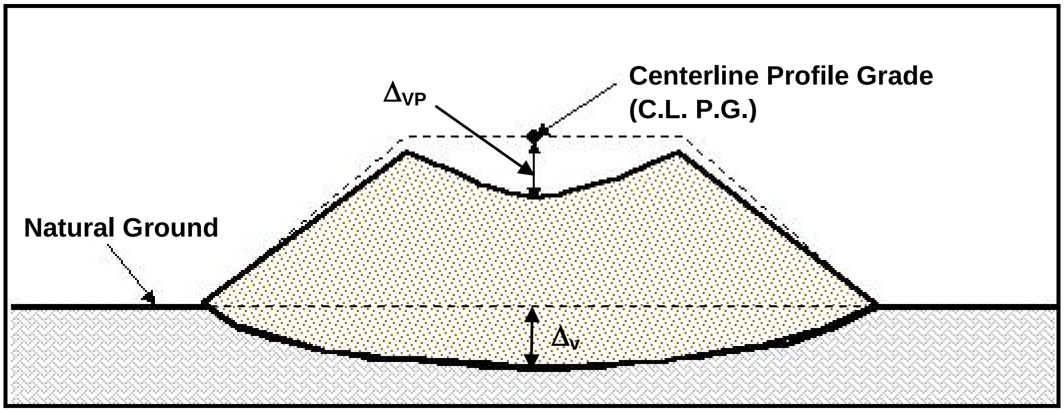

Figure 10-3, Embankment Settlement (Section A–A)

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.

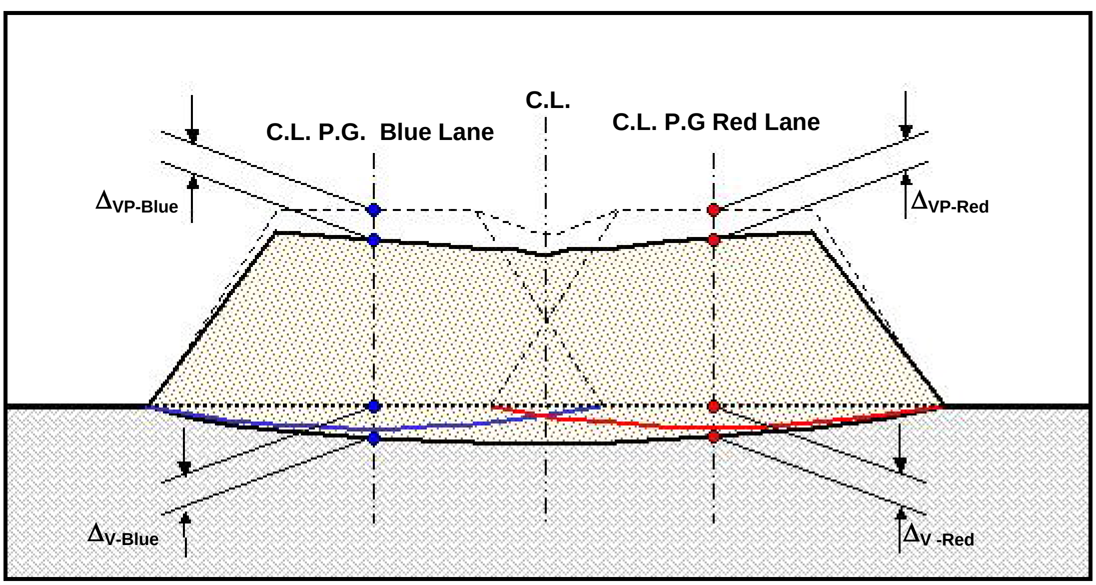

Figure 10-4, Divided Highway (Section A-A)

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.

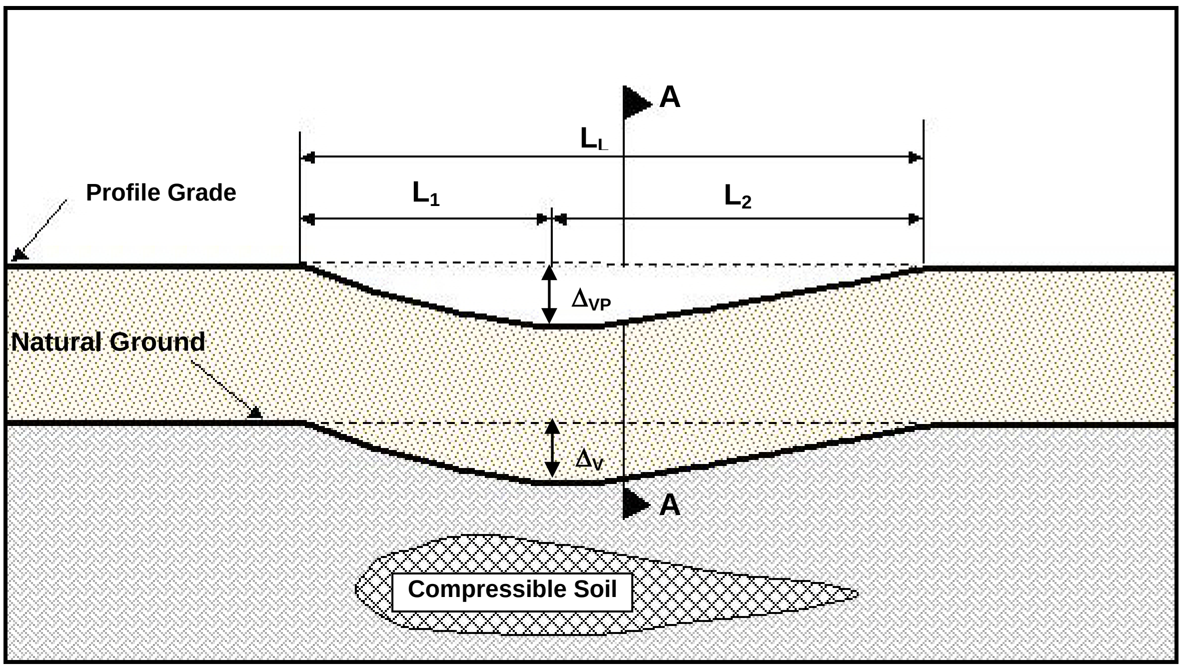

Figure 10-5, Embankment Settlement Profile

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.

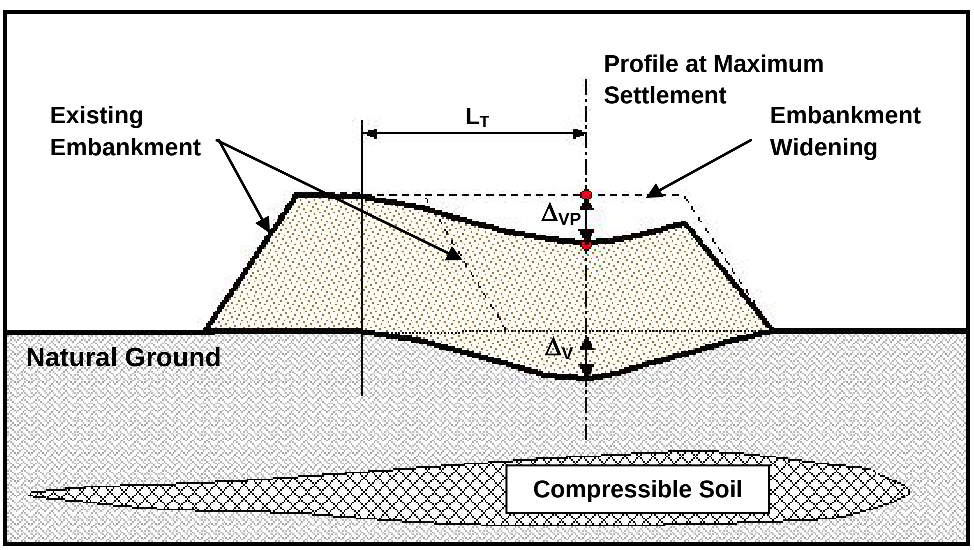

Figure 10-6, Embankment Widening Settlement (Section A-A)

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.

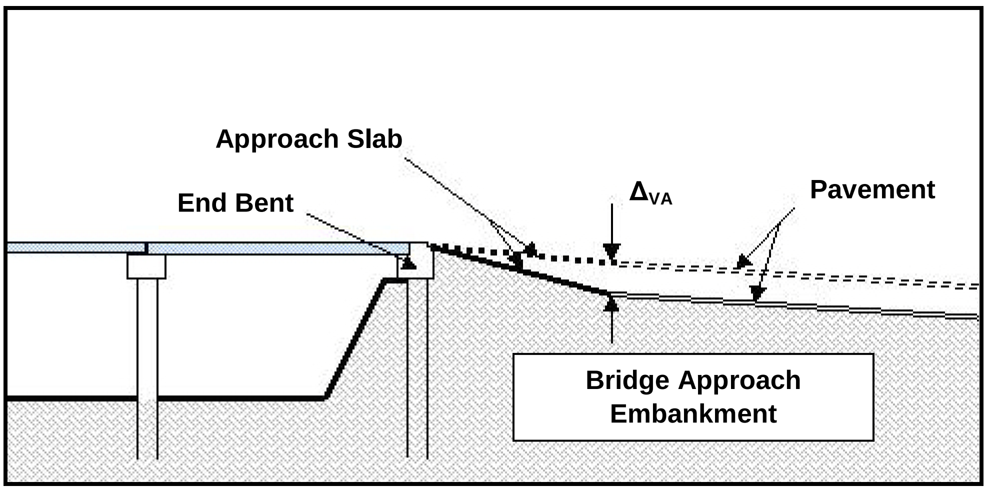

Figure 10-7, Bridge Approach Embankment Settlement

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.

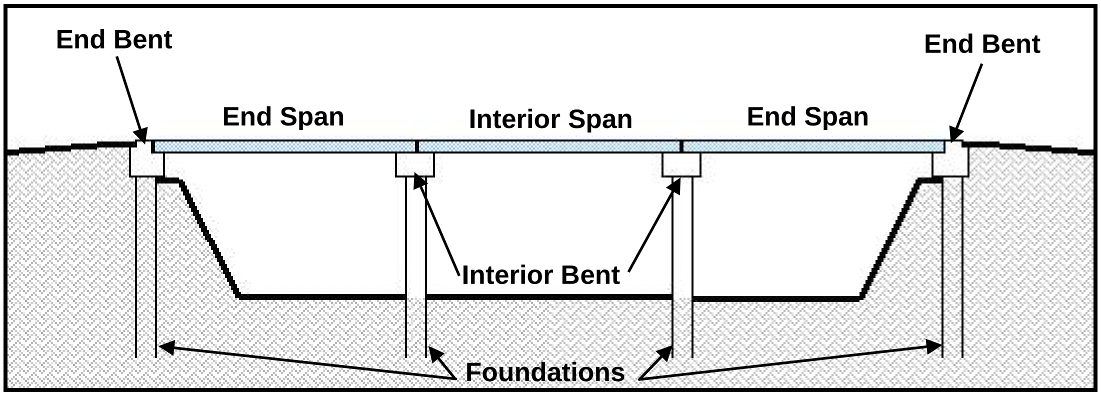

Figure 10-8, Bridge Layout (Simple or Continuous Span)

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).

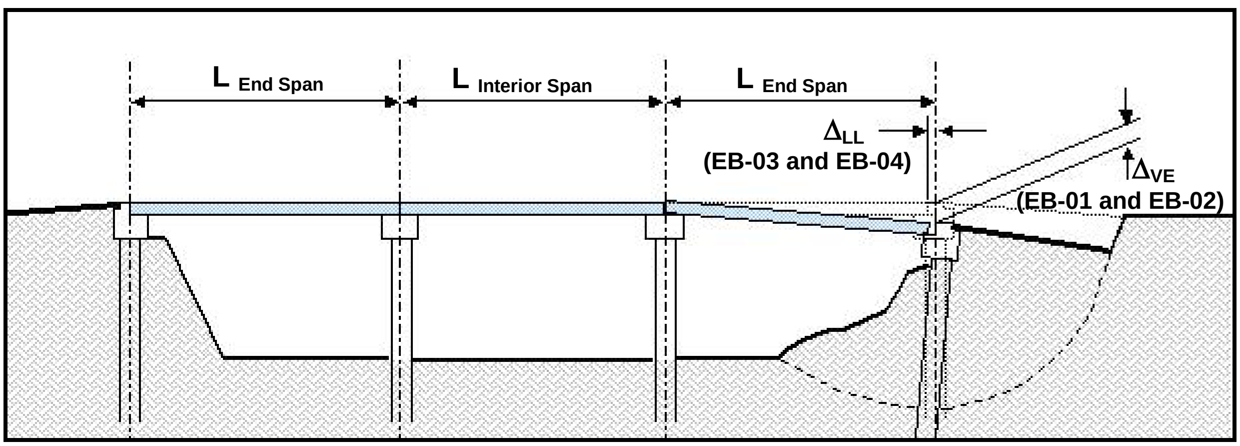

Figure 10-9, Bridge End Bent Slope Instability Deformation

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 ($L_{Span} = L_{Interior\ Span}$).

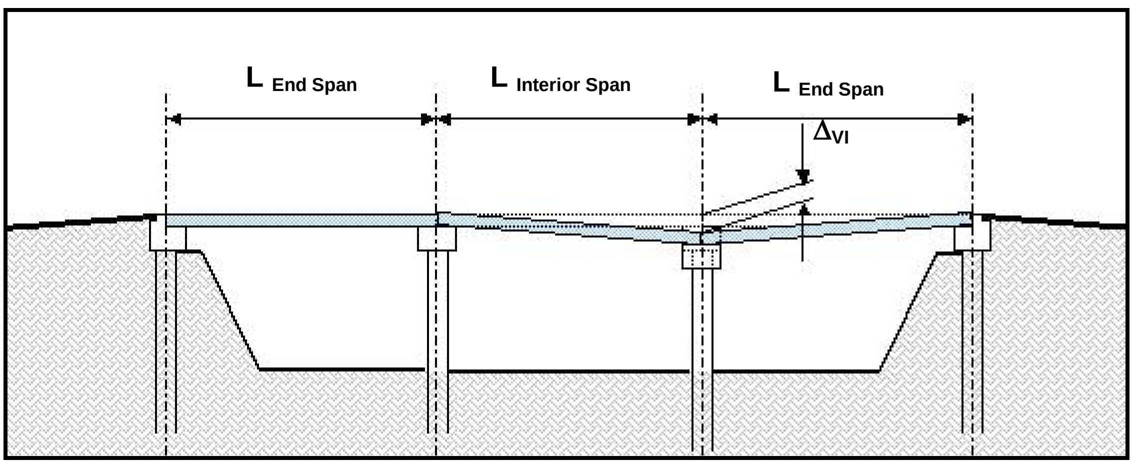

Figure 10-10, Bridge Interior Bent Settlement

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.

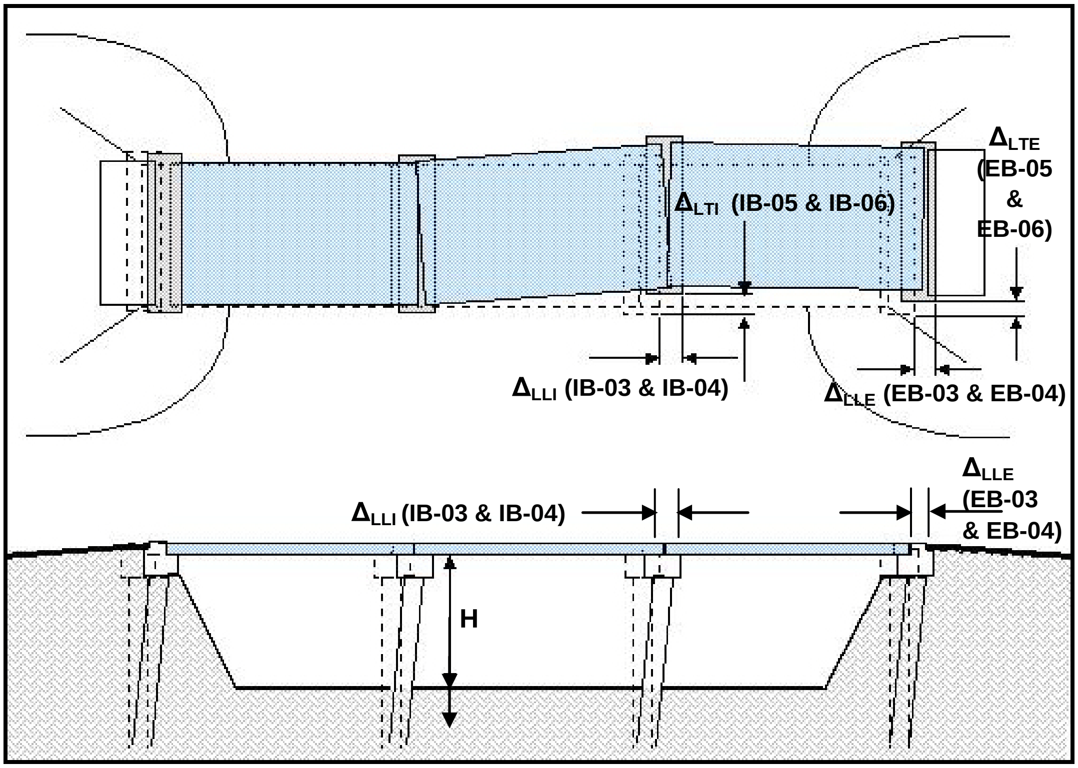

Figure 10-11, Bridge Lateral Displacement

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.

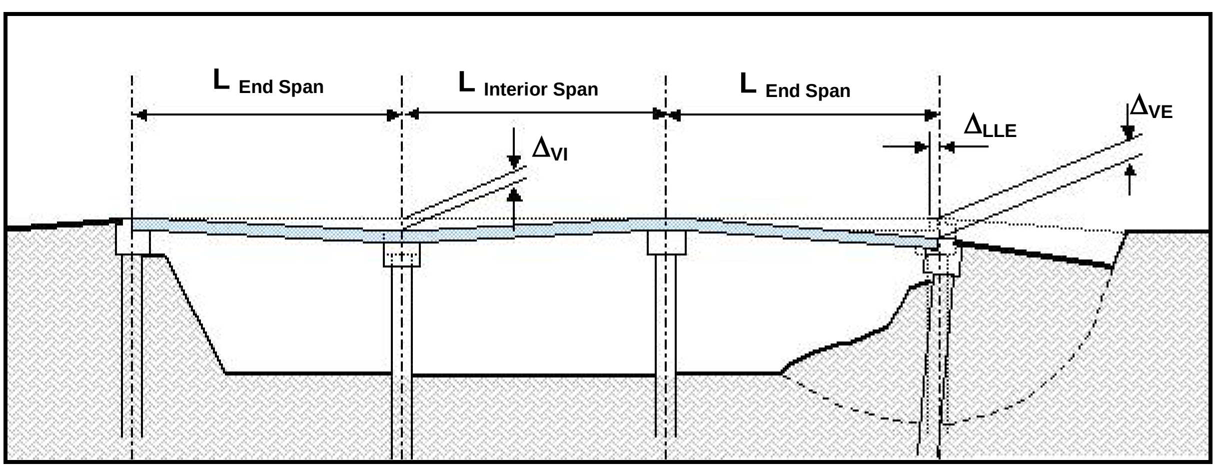

Figure 10-12, Continuous Bridge Settlements

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.

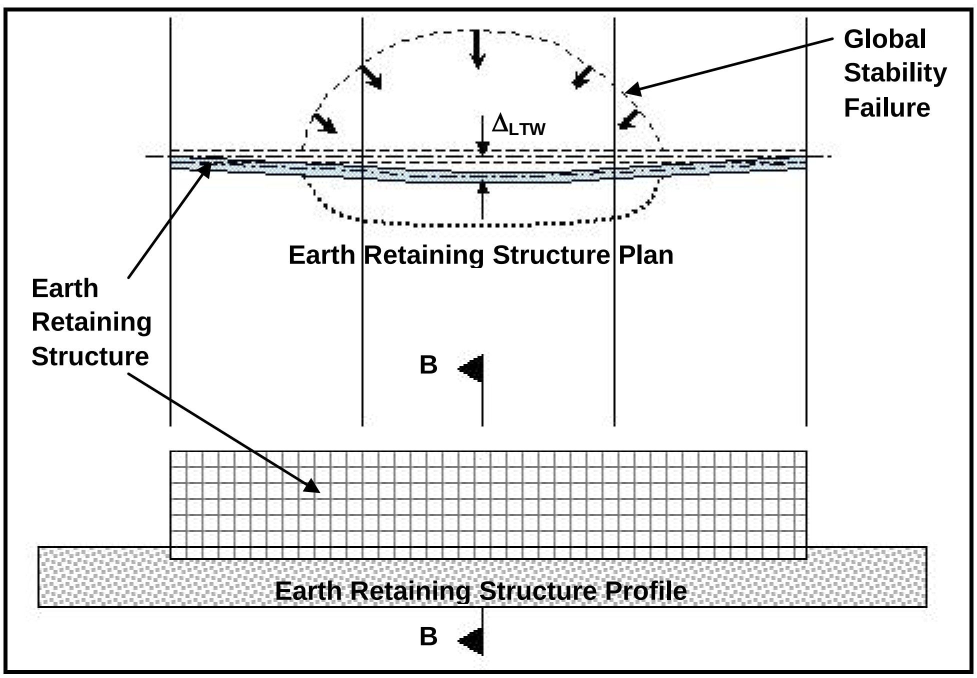

Figure 10-13, ERS Global Instability

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.

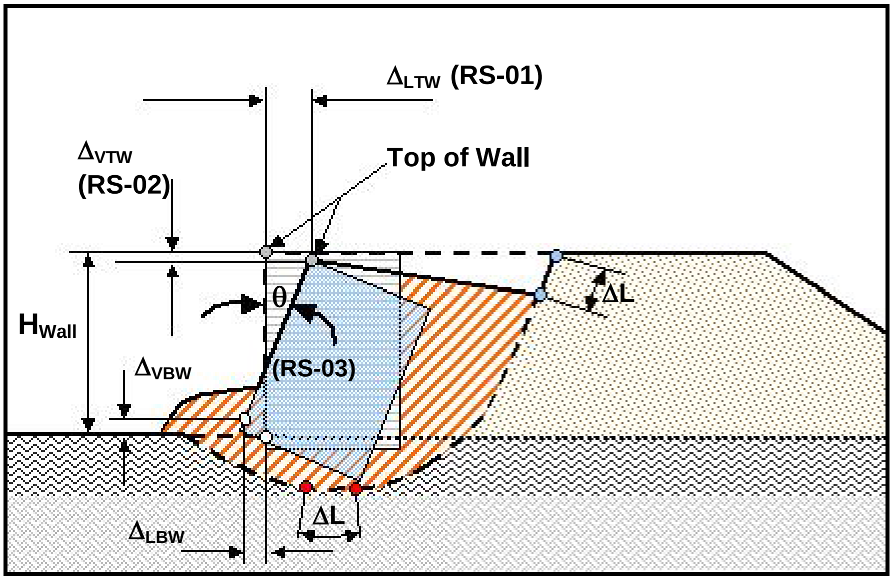

Figure 10-14, ERS Circular-Arc Instability (Section B-B)

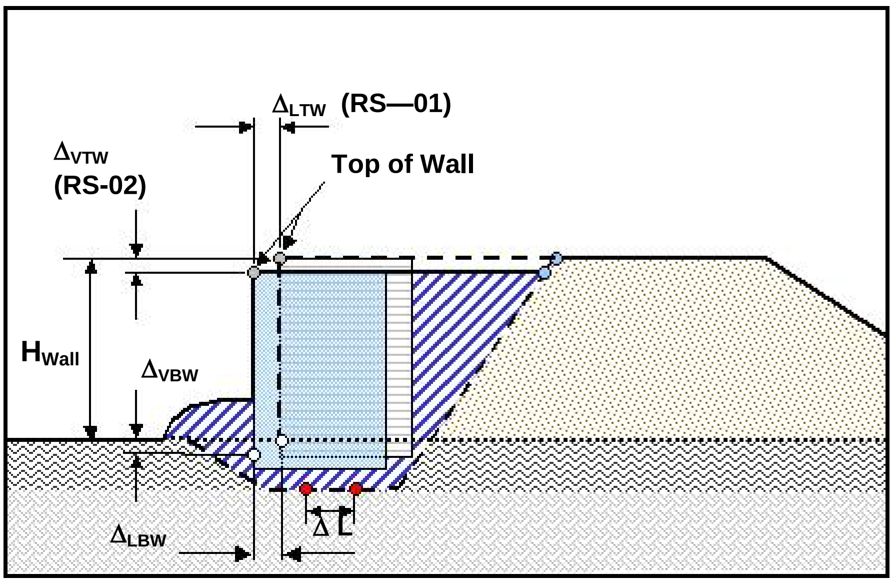

Figure 10-15, ERS Sliding-Wedge Instability (Section B-B)

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.

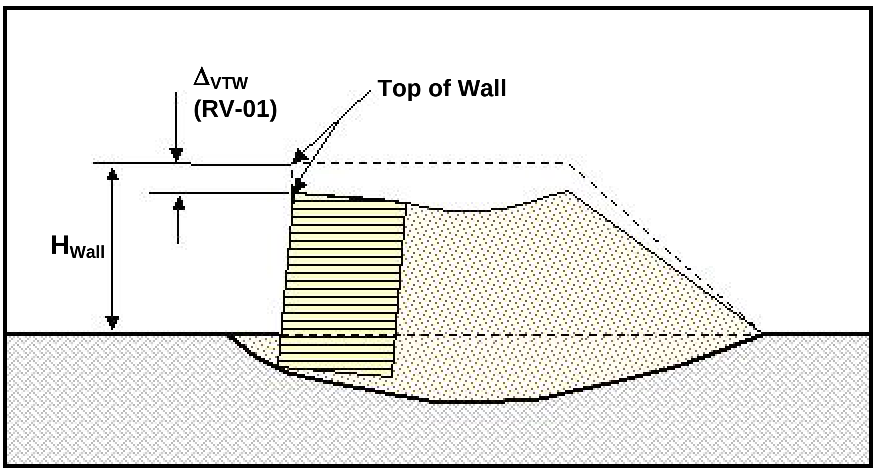

Figure 10-16, ERS Settlement (Section B–B)

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.

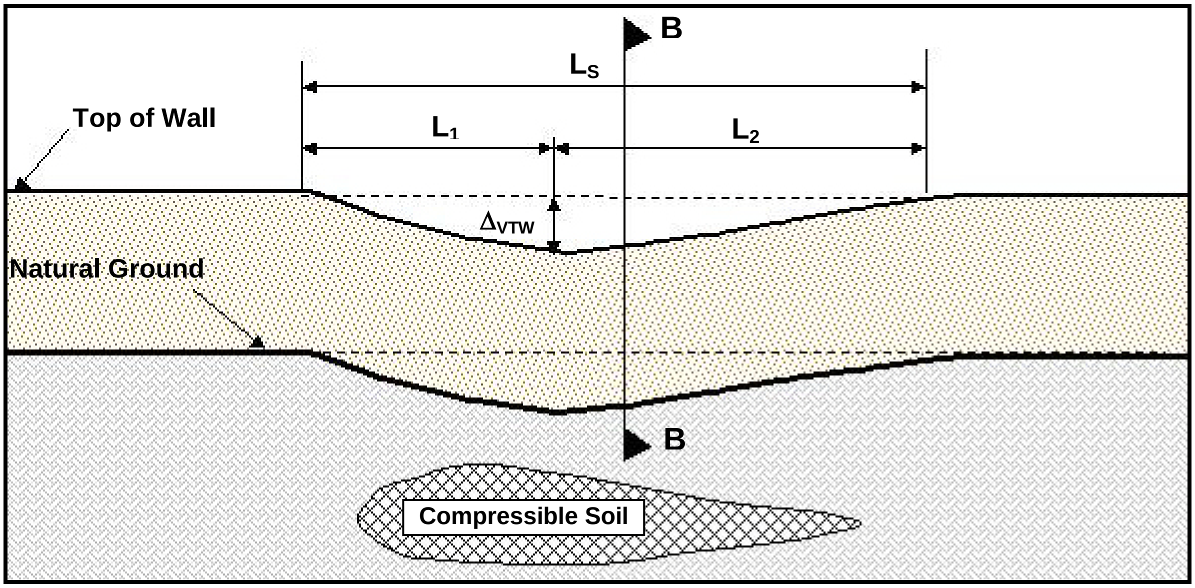

Figure 10-17, ERS Settlement Profile

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.

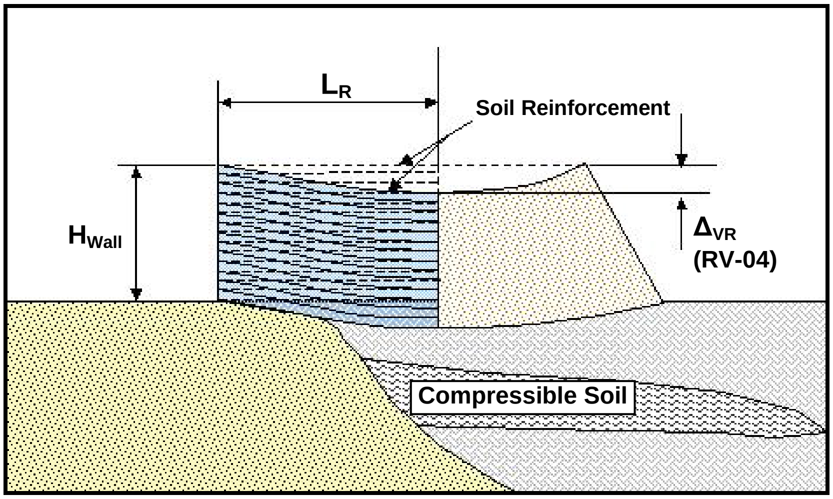

Figure 10-18, ERS Reinforced Soils - Transverse Differential Settlement

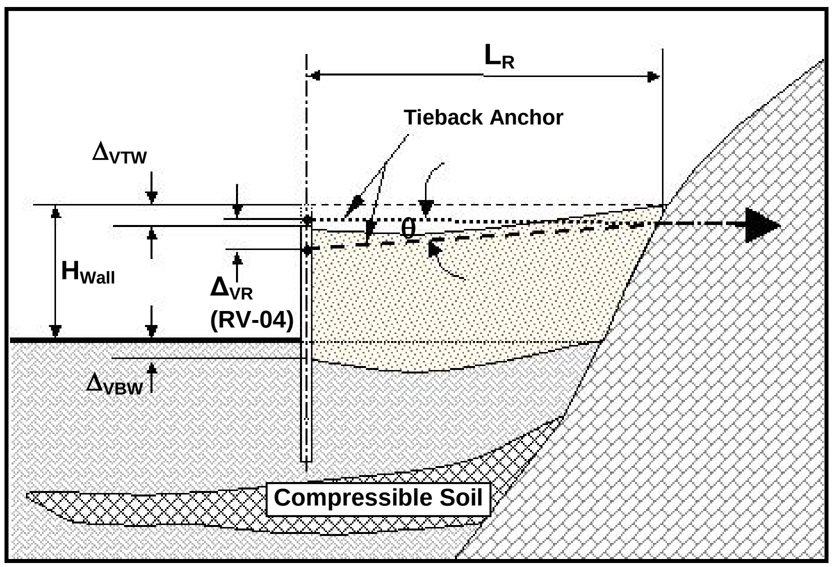

Figure 10-19, ERS Tieback Anchor - Transverse Differential Settlement

10.6.5 <u>Lateral Displacements</u>

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.

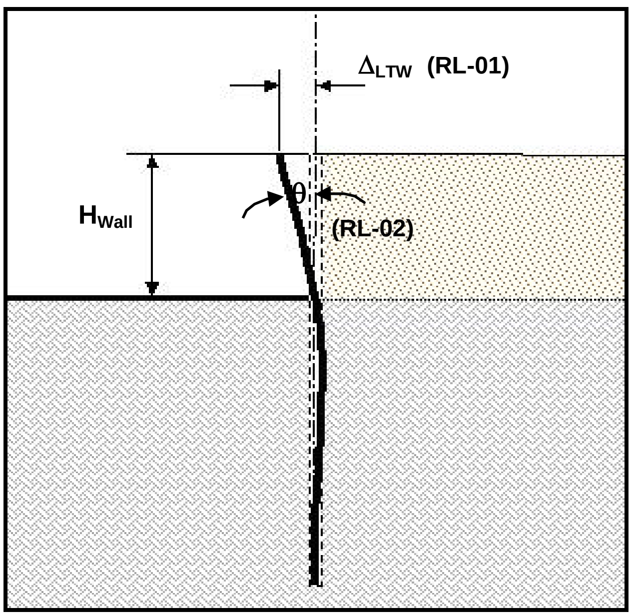

Figure 10-20, Cut ERS Section C-C Lateral Deformations

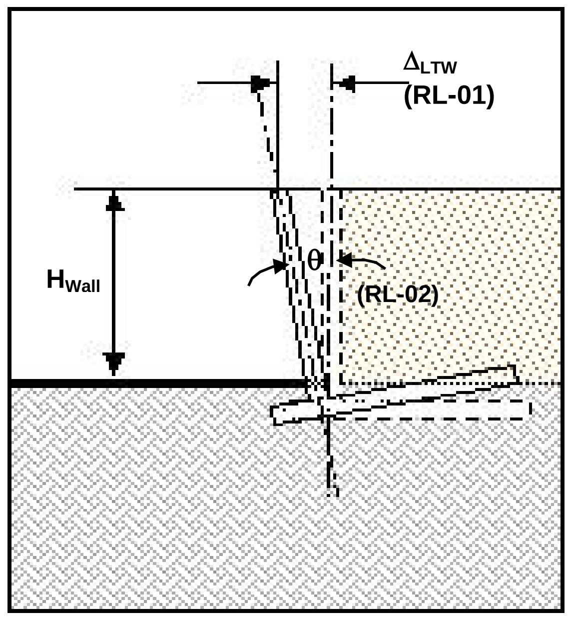

Figure 10-21, Fill ERS Section C-C Lateral Deformations

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.

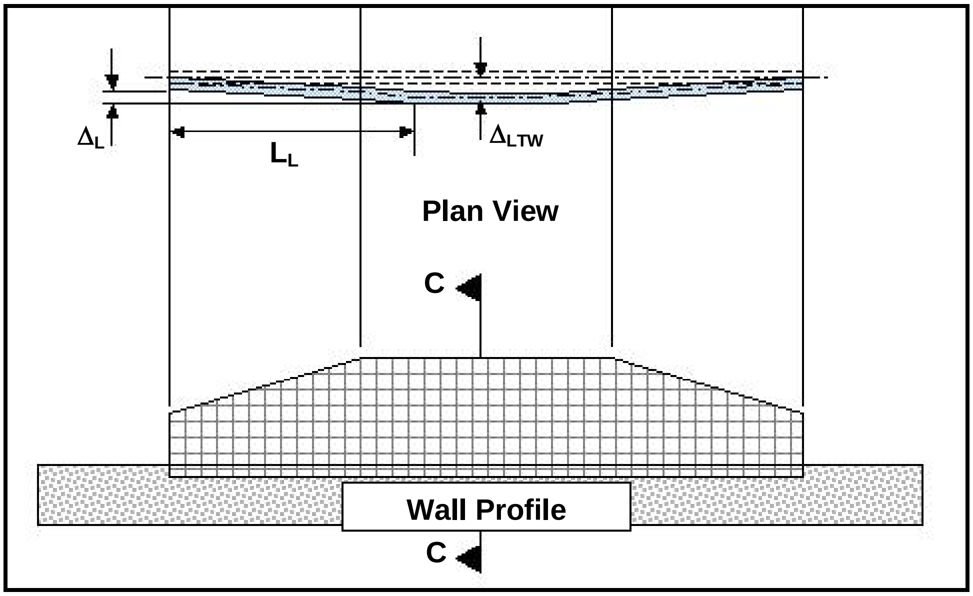

Figure 10-22, ERS Lateral Deformations

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

Design Earthquake	Performance Level	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

10.7.2.2 Performance Limits

Table 10-28, Embankment Global Instability Performance Limits at EE I Limit State

Deformation ID No.		EE I Limit State Performance Limit Description (1)	Design EQ	ROC		
				I	II	III
Vertical Displacement	**GI-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”
	GI -02	Maximum Vertical Displacement at bottom of the slope failure surface. (Inches)	FEE	1.00”	2.00”	4.00”
			SEE	2.00”	4.00”	8.00”
Lateral (2) Displacement	**GI-03**	Maximum Lateral Displacement at top of the slope failure surface. (Inches)	FEE	3.00”	6.00”	24.00”
			SEE	4.00”	12.00”	60.00”
	GI-04	Maximum Lateral Displacement at bottom of the slope failure surface. (Inches)	FEE	3.00”	6.00”	24.00”
			SEE	4.00”	12.00”	60.00”

(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	II	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	II	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	II	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)

<table>
<tr><th colspan="2" rowspan="2">Deformation ID No.</th><th rowspan="2">Extreme Event I Performance Limit Description</th><th rowspan="2">Design EQ</th><th colspan="3">OC</th></tr>
<tr><th>I</th><th>II</th><th>III</th></tr>
<tr><td rowspan="12">Bridge End Bents</td><td rowspan="2">EB–01</td><td rowspan="2">Maximum Vertical Differential Settlement for Integral/Semi-Integral End Bent (Inches) (1)</td><td>FEE</td><td>0.020 L_{Span}</td><td>0.020 L_{Span}</td><td>0.020 L_{Span}</td></tr>
<tr><td>SEE</td><td>0.040 L_{Span}</td><td>0.040 L_{Span}</td><td>0.040 L_{Span}</td></tr>
<tr><td rowspan="2">EB–02</td><td rowspan="2">Maximum Vertical Differential Settlement for Free Standing End Bent (Inches) (1)</td><td>FEE</td><td>0.040 L_{Span}</td><td>0.040 L_{Span}</td><td>0.040 L_{Span}</td></tr>
<tr><td>SEE</td><td>0.080 L_{Span}</td><td>0.080 L_{Span}</td><td>0.080 L_{Span}</td></tr>
<tr><td rowspan="2">EB–03</td><td rowspan="2">Maximum Lateral Longitudinal Displacement for Integral/Semi-Integral End Bent (Inches) (2)</td><td>FEE</td><td>2"</td><td>4"</td><td>12"</td></tr>
<tr><td>SEE</td><td>4"</td><td>8"</td><td>12"</td></tr>
<tr><td rowspan="2">EB–04</td><td rowspan="2">Maximum Lateral Longitudinal Displacement for Free Standing End Bent (Inches) (2)</td><td>FEE</td><td>1"</td><td>2"</td><td>8"</td></tr>
<tr><td>SEE</td><td>3"</td><td>6"</td><td>8"</td></tr>
<tr><td rowspan="2">EB–05</td><td rowspan="2">Maximum Lateral Transverse Displacement Integral/Semi-Integral End Bent (Inches) (2)</td><td>FEE</td><td>2"</td><td>4"</td><td>12"</td></tr>
<tr><td>SEE</td><td>4"</td><td>8"</td><td>12”</td></tr>
<tr><td rowspan="2">EB–06</td><td rowspan="2">Maximum Lateral Transverse Displacement for Free Standing End Bent (Inches) (2)</td><td>FEE</td><td>2"</td><td>4"</td><td>12"</td></tr>
<tr><td>SEE</td><td>4"</td><td>8"</td><td>12”</td></tr>
</table>

(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	ROC II	ROC 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

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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.

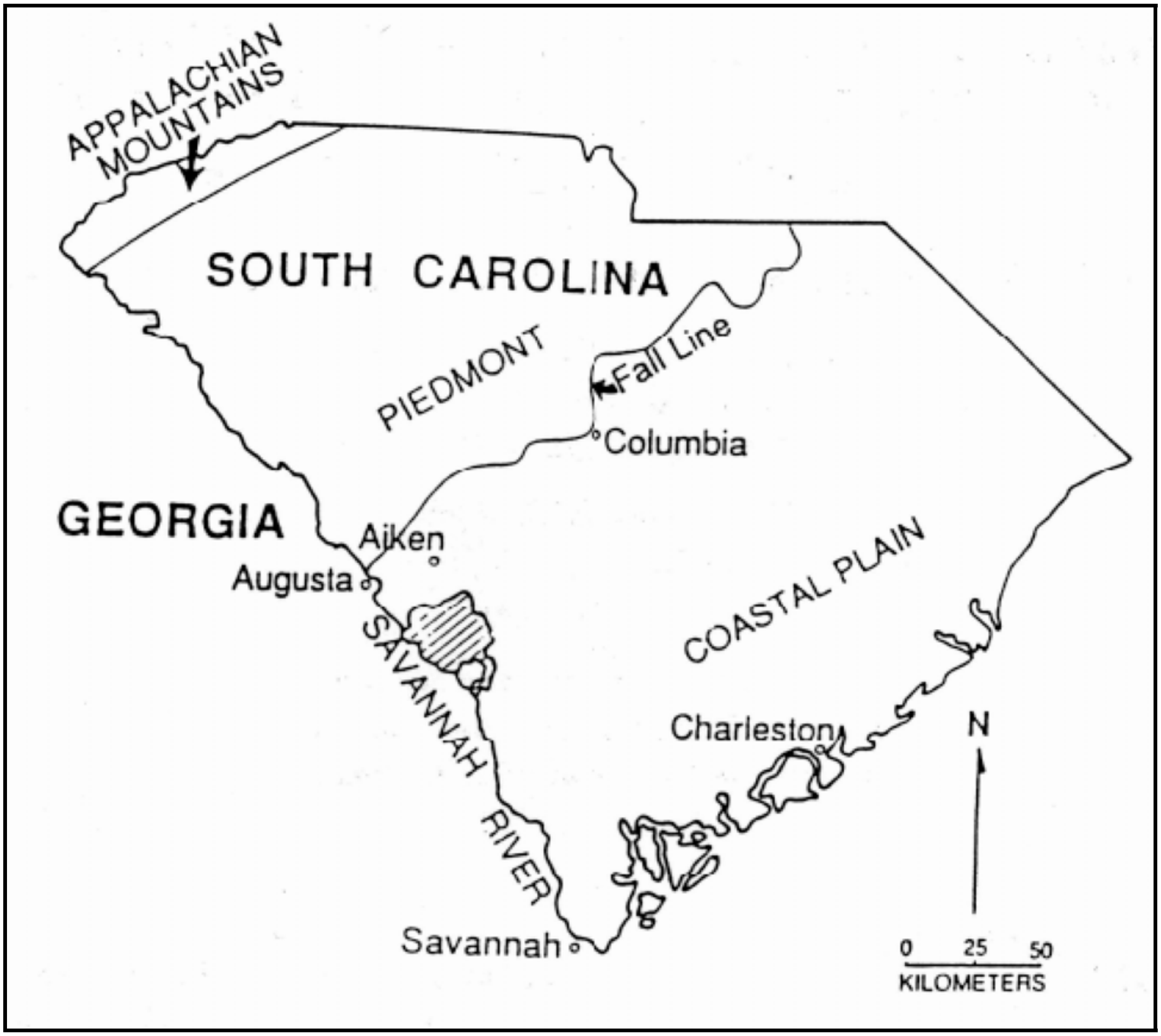


Figure 11-1, South Carolina Physiographic Units (Snipes et al., 1993)

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.

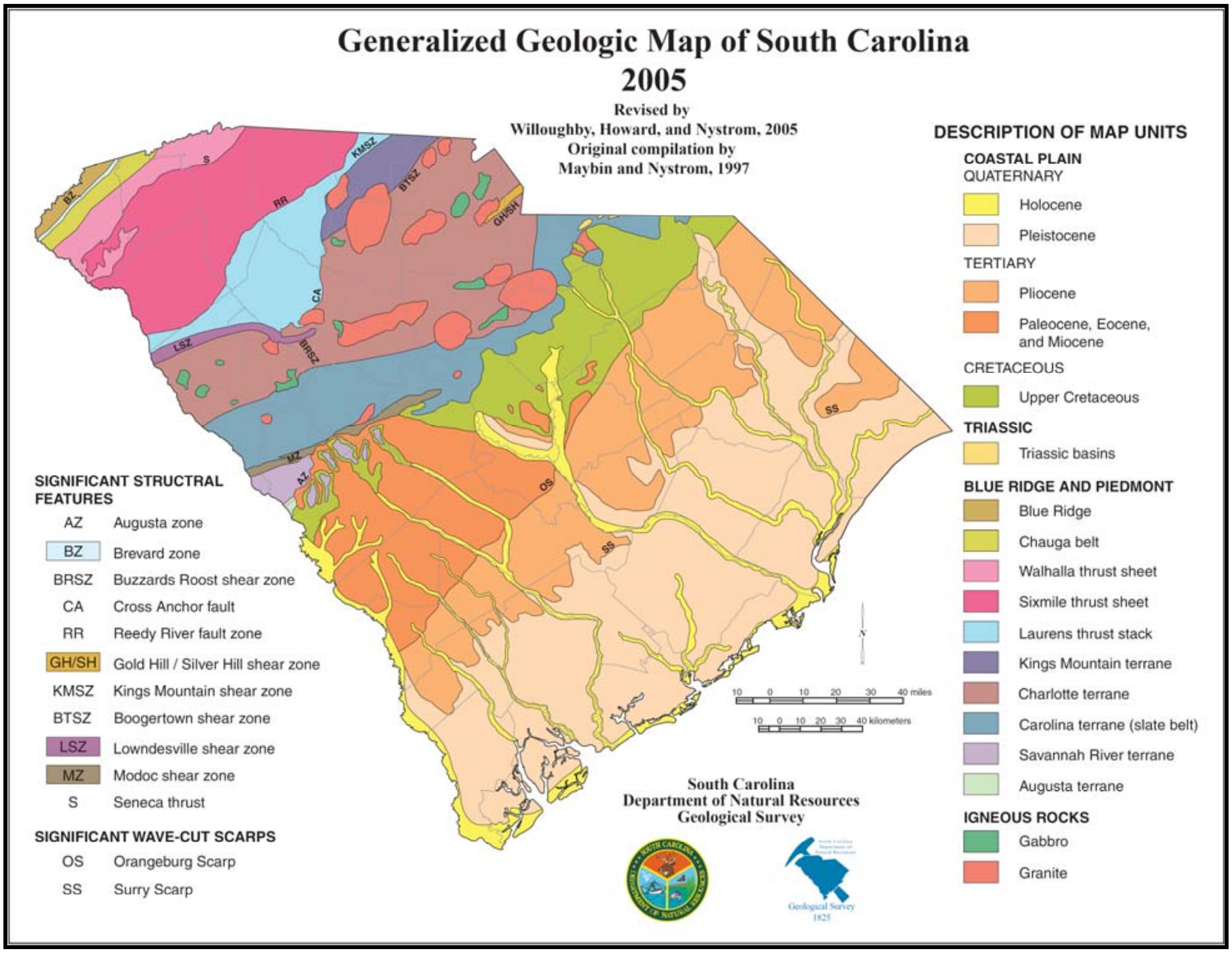
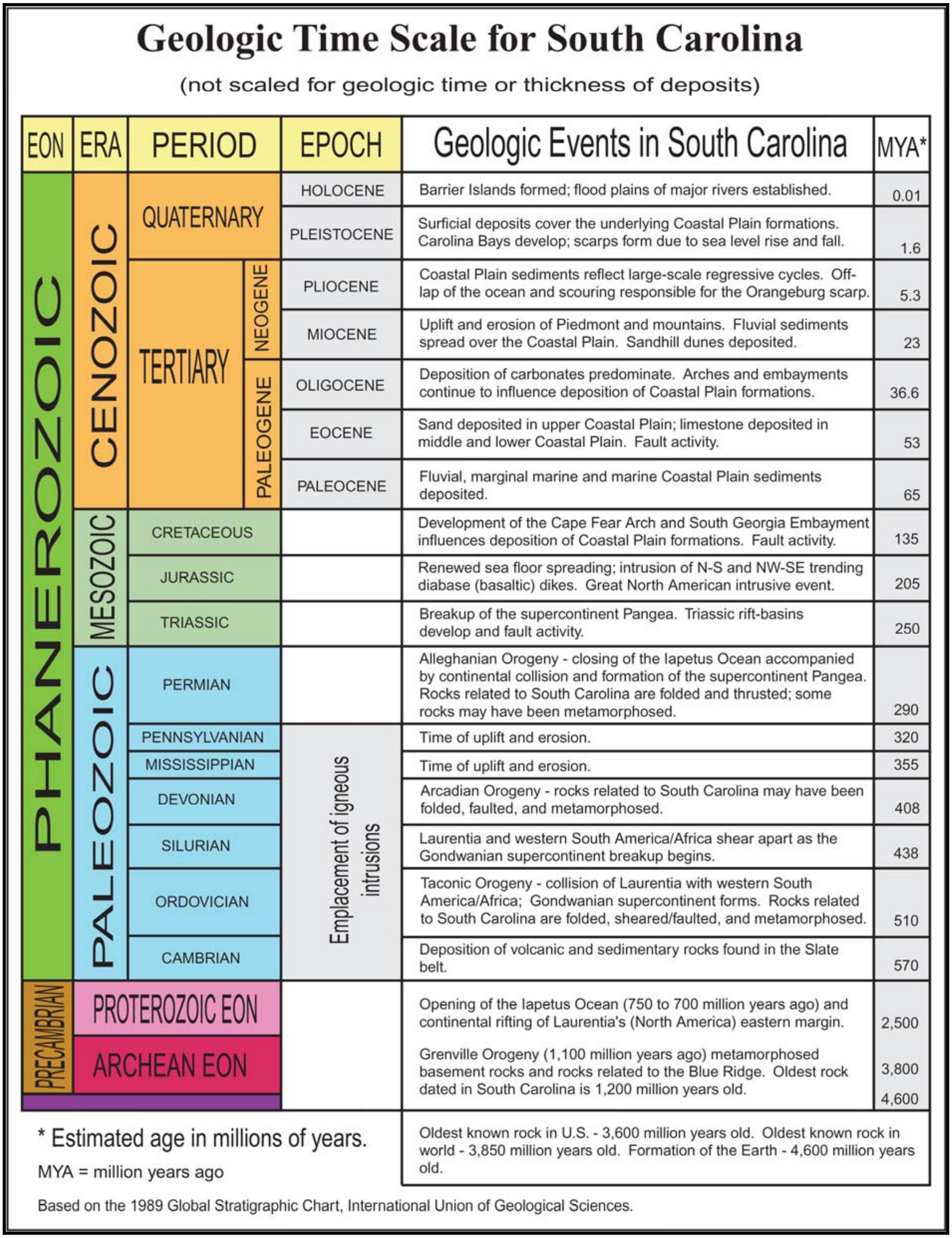


Figure 11-2, 2005 Generalized Geologic Map of South Carolina, (SCDNR)

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
				PLEISTOCENE	Surficial deposits cover the underlying Coastal Plain formations. Carolina Bays develop; scarps form due to sea level rise and fall.	1.6
		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
				MIOCENE	Uplift and erosion of Piedmont and mountains. Fluvial sediments spread over the Coastal Plain. Sandhill dunes deposited.	23
			PALEOGENE	OLIGOCENE	Deposition of carbonates predominate. Arches and embayments continue to influence deposition of Coastal Plain formations.	36.6
				EOCENE	Sand deposited in upper Coastal Plain; limestone deposited in middle and lower Coastal Plain. Fault activity.	53
				PALEOCENE	Fluvial, marginal marine and marine Coastal Plain sediments deposited.	65
	MESOZOIC	CRETACEOUS			Development of the Cape Fear Arch and South Georgia Embayment influences deposition of Coastal Plain formations. Fault activity.	135
		JURASSIC			Renewed sea floor spreading; intrusion of N-S and NW-SE trending diabase (basaltic) dikes. Great North American intrusive event.	205
		TRIASSIC			Breakup of the supercontinent Pangea. Triassic rift-basins develop and fault activity.	250
	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
		PENNSYLVANIAN		Emplacement of igneous intrusions	Time of uplift and erosion.	320
		MISSISSIPPIAN			Time of uplift and erosion.	355
		DEVONIAN			Arcadian Orogeny - rocks related to South Carolina may have been folded, faulted, and metamorphosed.	408
		SILURIAN			Laurentia and western South America/Africa shear apart as the Gondwanian supercontinent breakup begins.	438
		ORDOVICIAN			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
		CAMBRIAN			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
	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
					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.	

* Estimated age in millions of years.

MYA = million years ago

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.

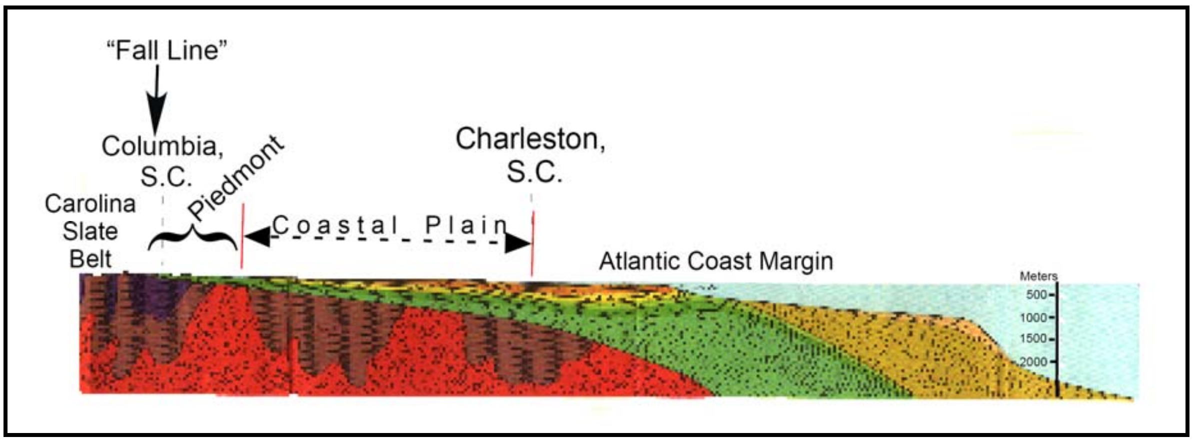


Figure 11-4, South Carolina "Fall Line"
(Odum et al., 2003)

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.

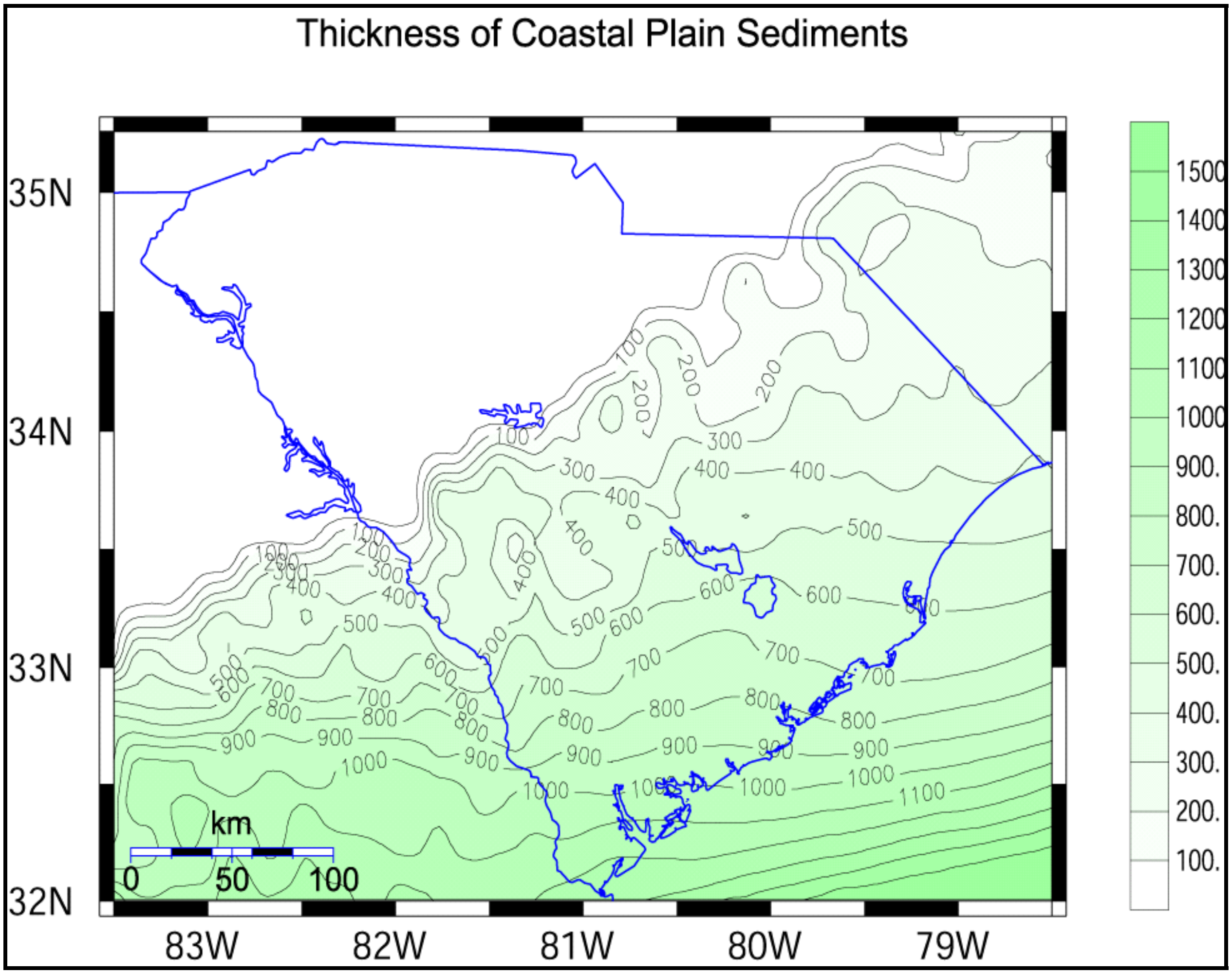


Figure 11-5, Contour Map of Coastal Plain Sediment Thickness, in meters (Chapman and Talwani, 2002)

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 Lower Coastal Plain

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 Middle Coastal Plain

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 Upper Coastal Plain

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.

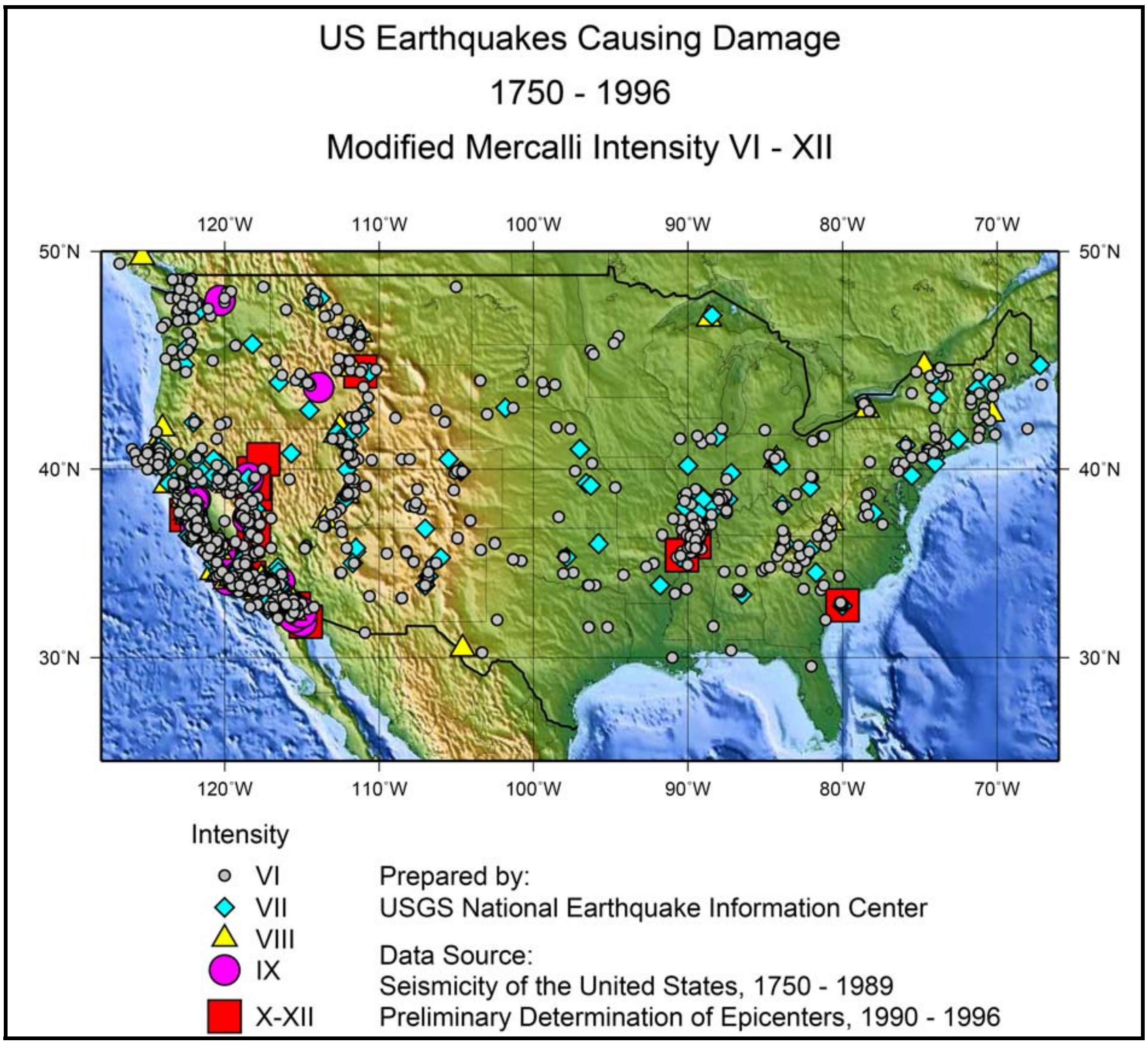


Figure 11-6, U.S. Earthquakes Causing Damage 1750 – 1996 (USGS)

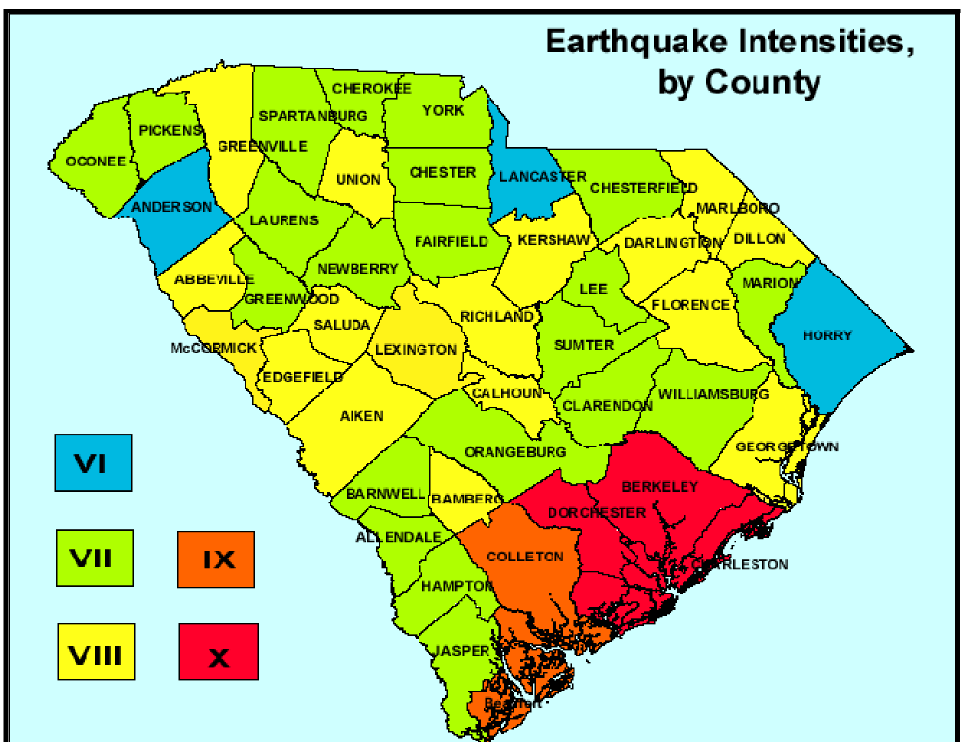
11.7.2 <u>SC Earthquake Intensity</u>

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.



(The Roman Numeral in the color key box indicates the earthquake intensity; X being the most intense.)

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.

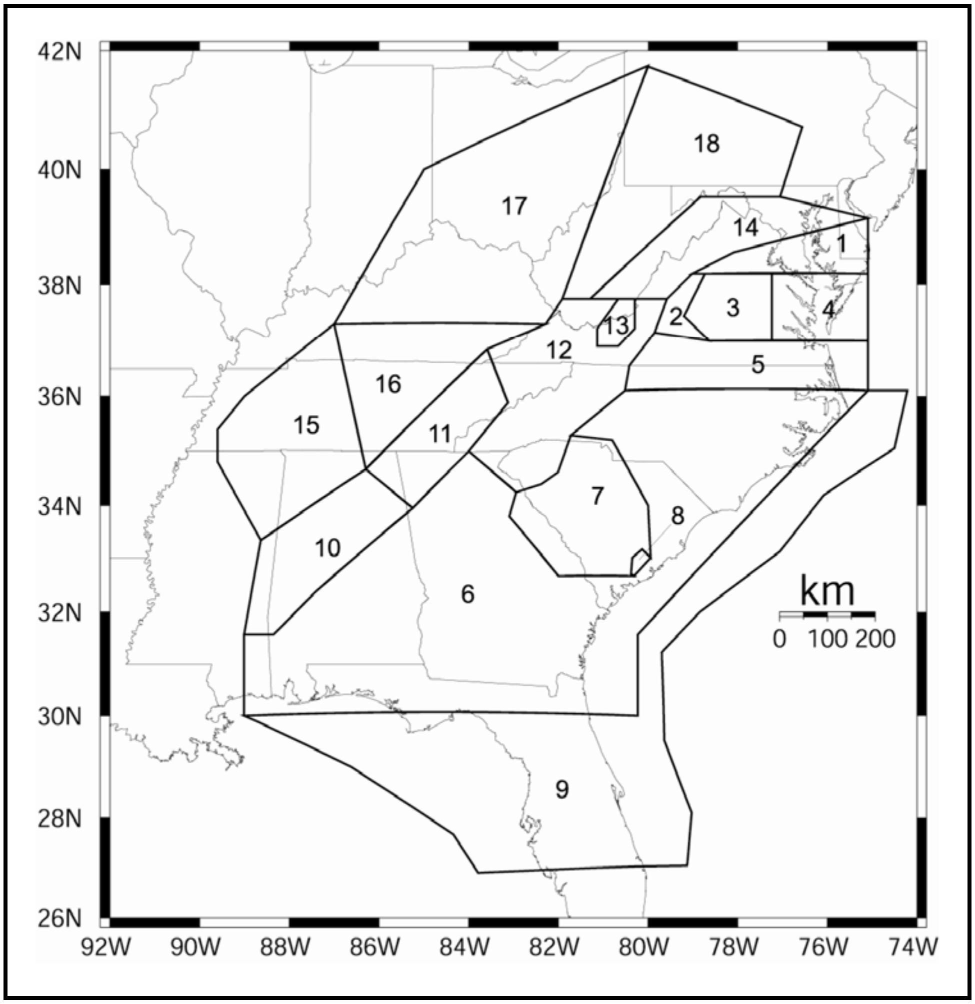


Figure 11-8, Source Areas for Non-Characteristic Earthquakes (Chapman and Talwani, 2002)

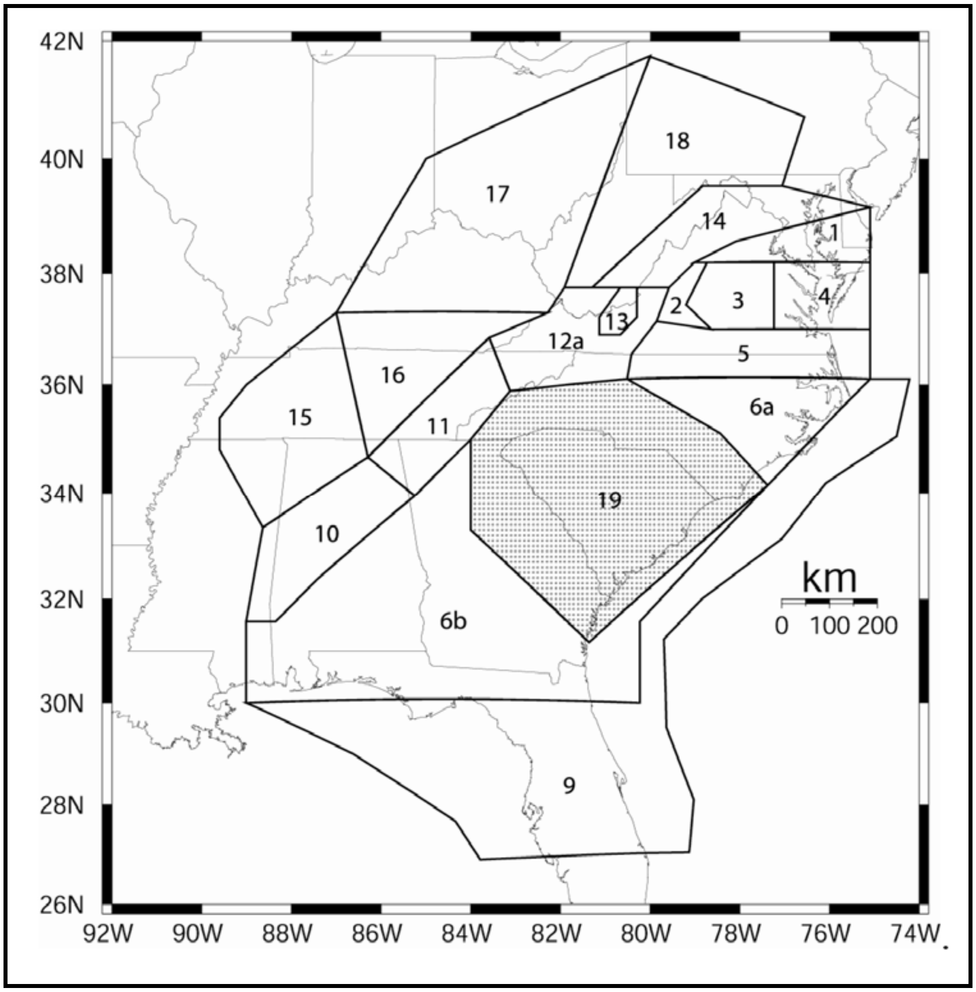


Figure 11-9, Alternative Source Areas for Non-Characteristic Earthquakes (Chapman and Talwani, 2002)

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.

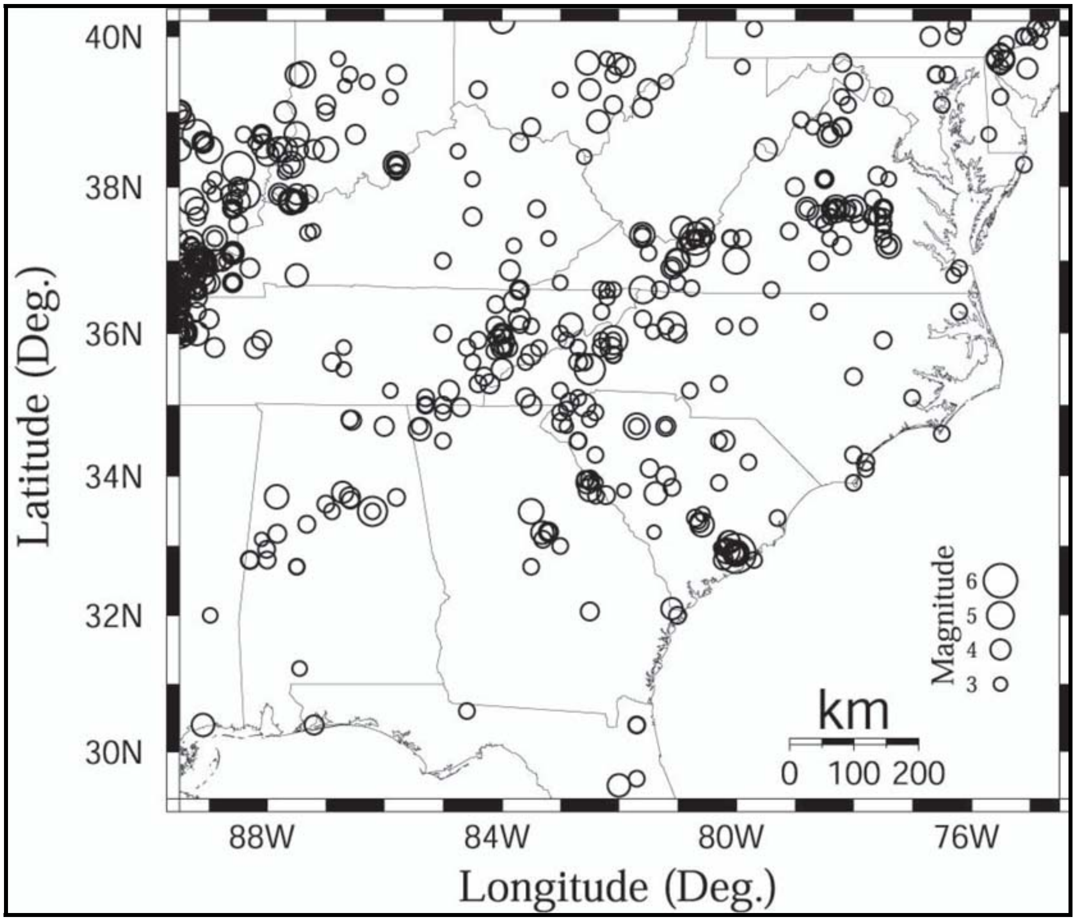


Figure 11-10, Southeastern U.S. Earthquakes (M_W > 3.0 from 1600 to Present) (Chapman and Talwani, 2002)

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.

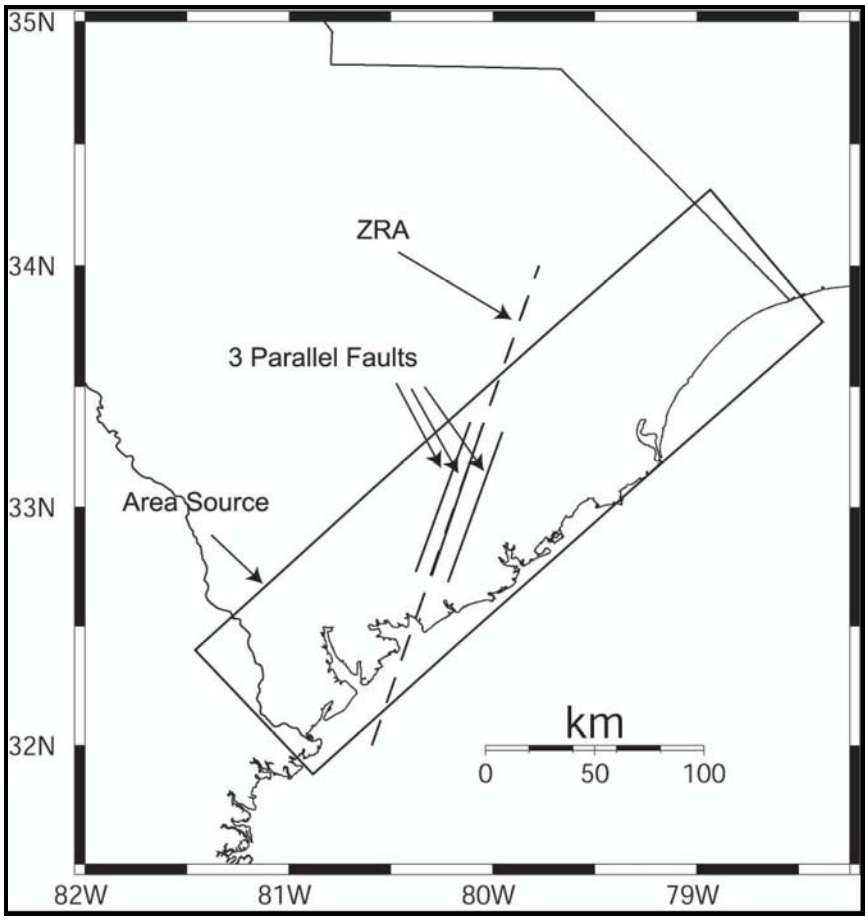


Figure 11-11, South Carolina Characteristic Earthquake Sources (Chapman and Talwani, 2002)

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.

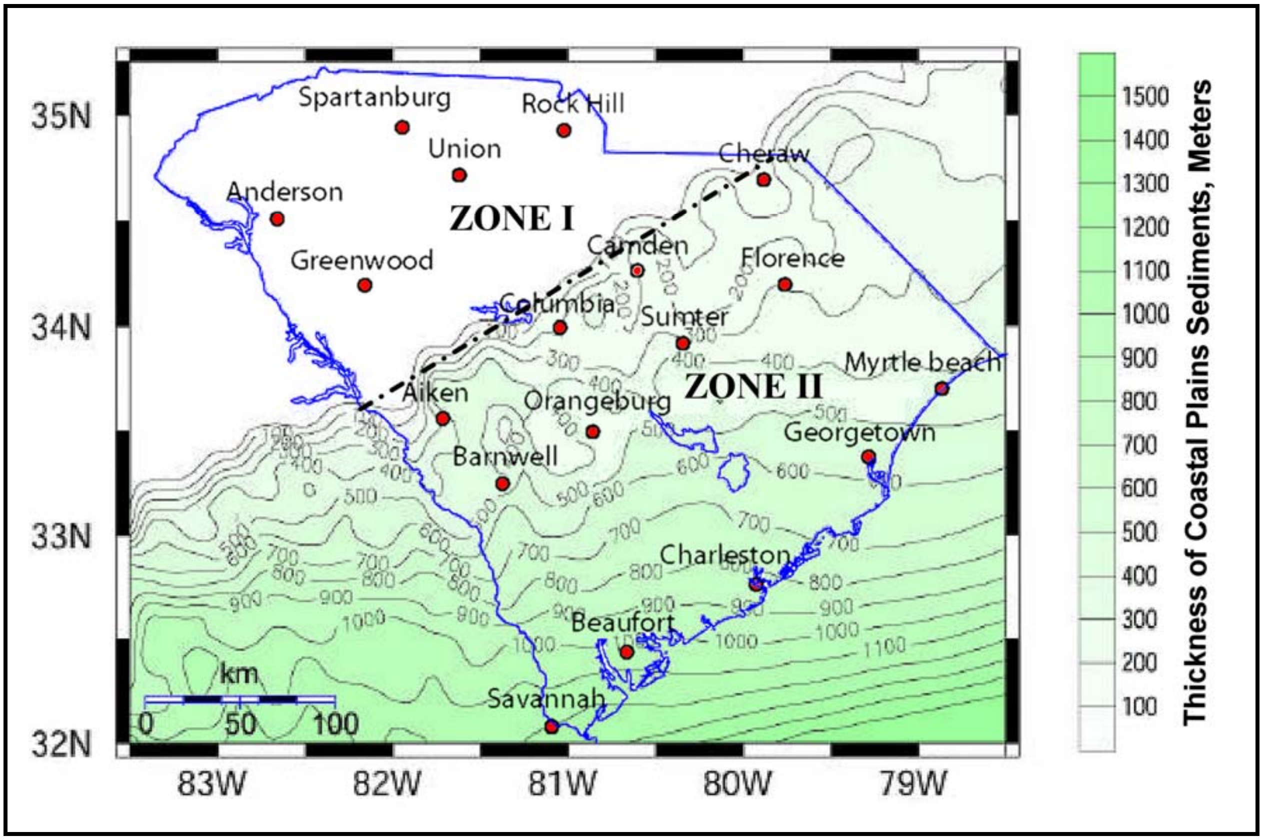


Figure 11-12, SCDOT Site Condition Selection Map (Modified Chapman and Talwani, 2002)

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>

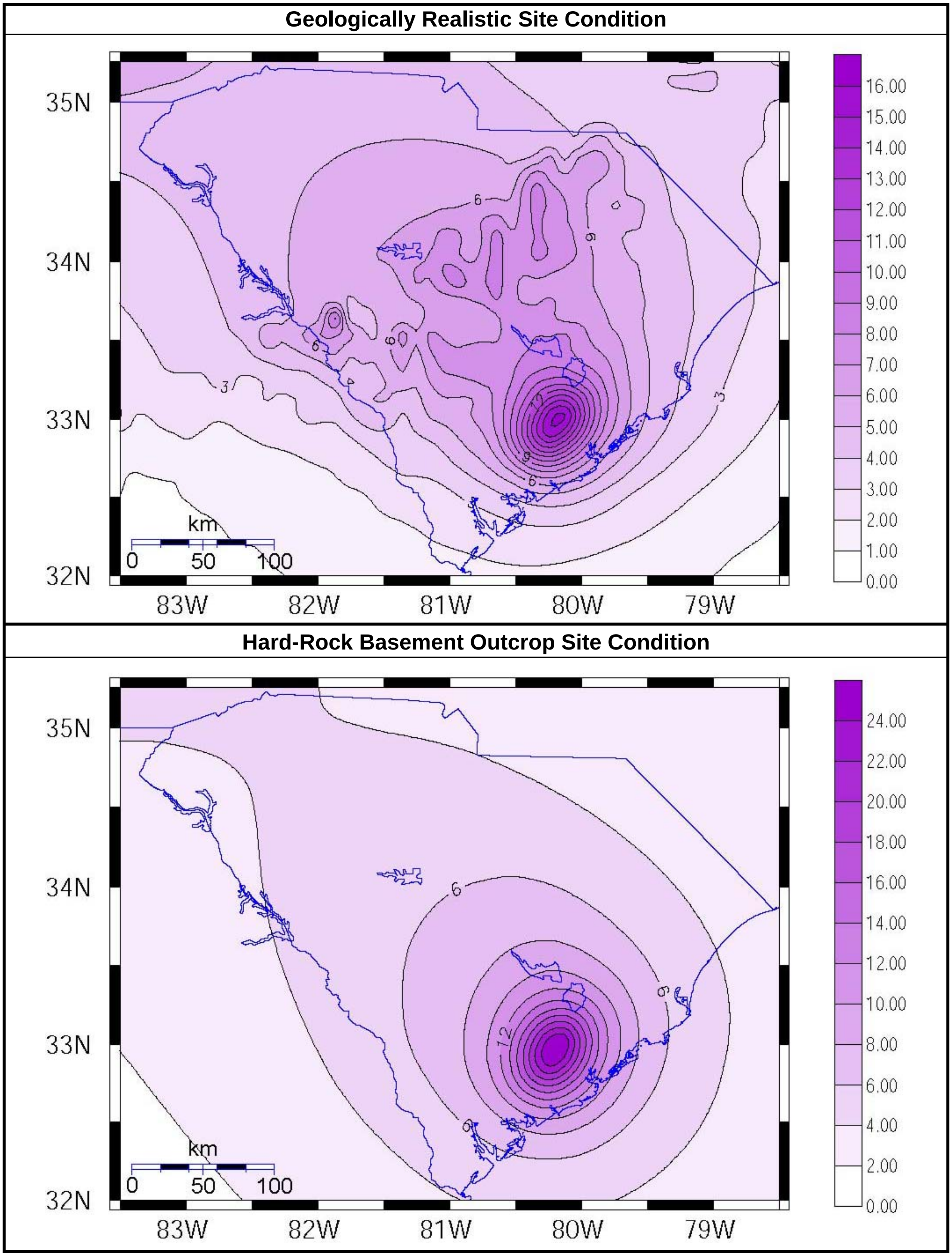


Figure 11-14, PGA (%g) - 15% P_E in 75 Years (10% P_E-50 Yr)
(Chapman and Talwani, 2002)

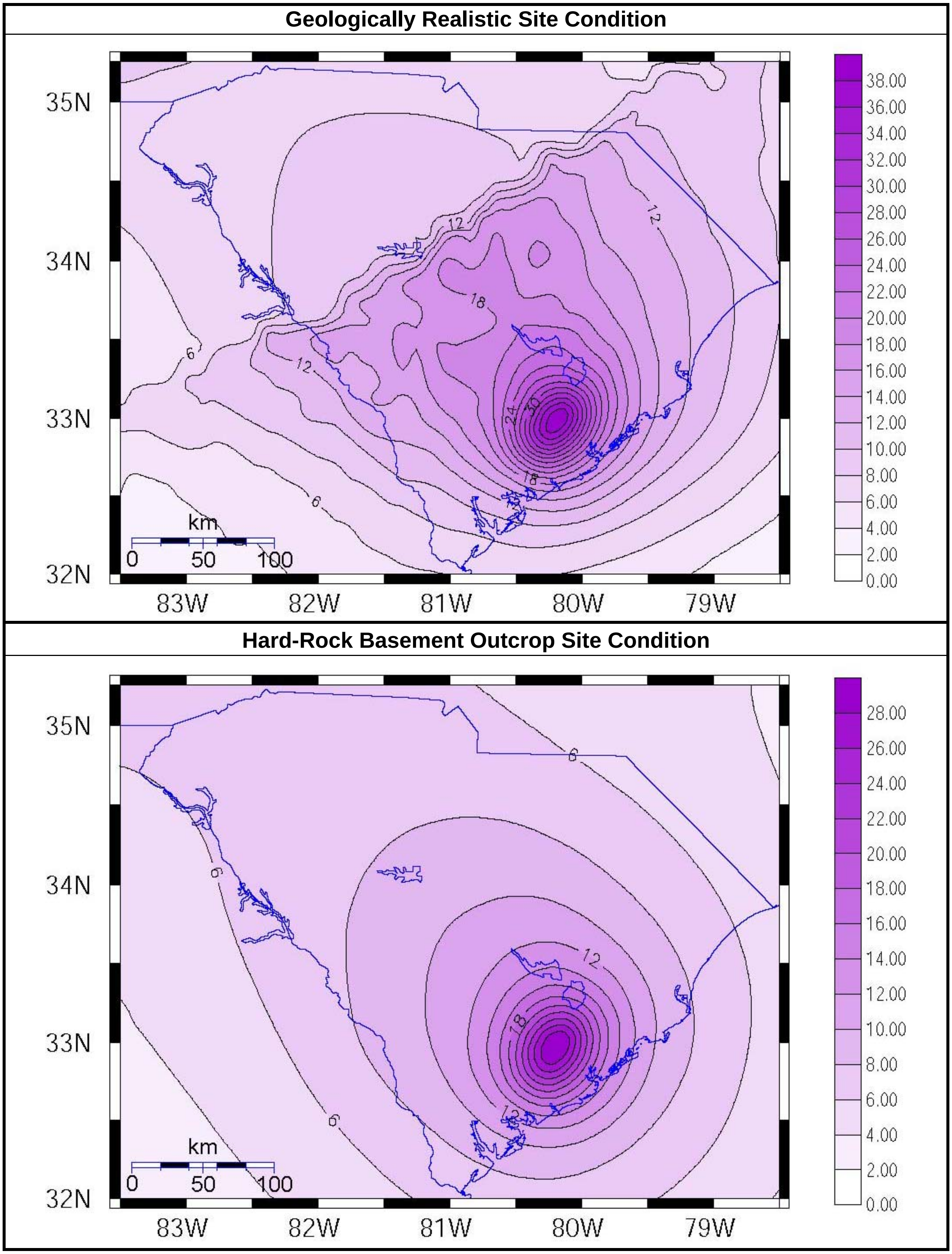


Figure 11-15, S_s Spectral Acceleration (%g) - 15% P_E in 75 Years (10% P_E-50 Yr) (Chapman and Talwani, 2002)

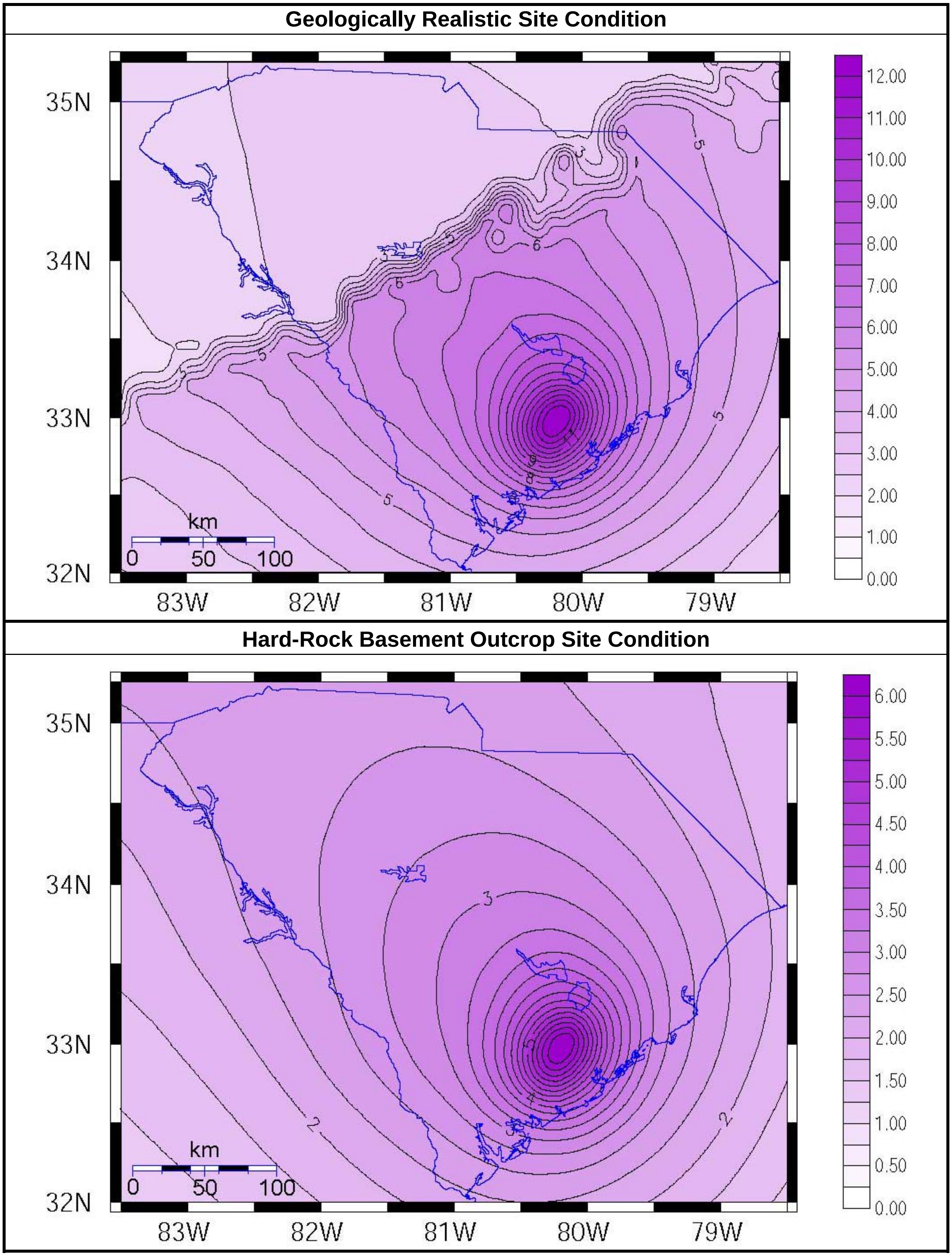
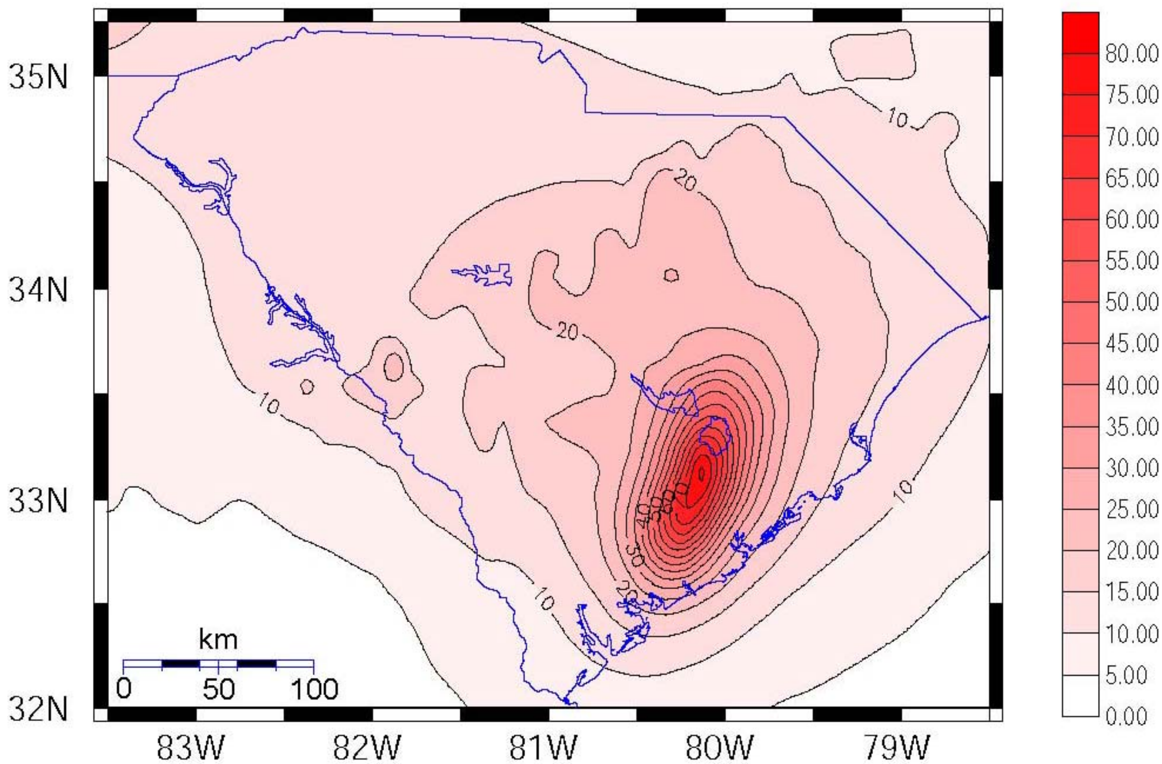


Figure 11-16, S_1 Spectral Acceleration (%g) - 15% P_E in 75 Years (10% P_E-50 Yr) (Chapman and Talwani, 2002)

Geologically Realistic Site Condition



Hard-Rock Basement Outcrop Site Condition

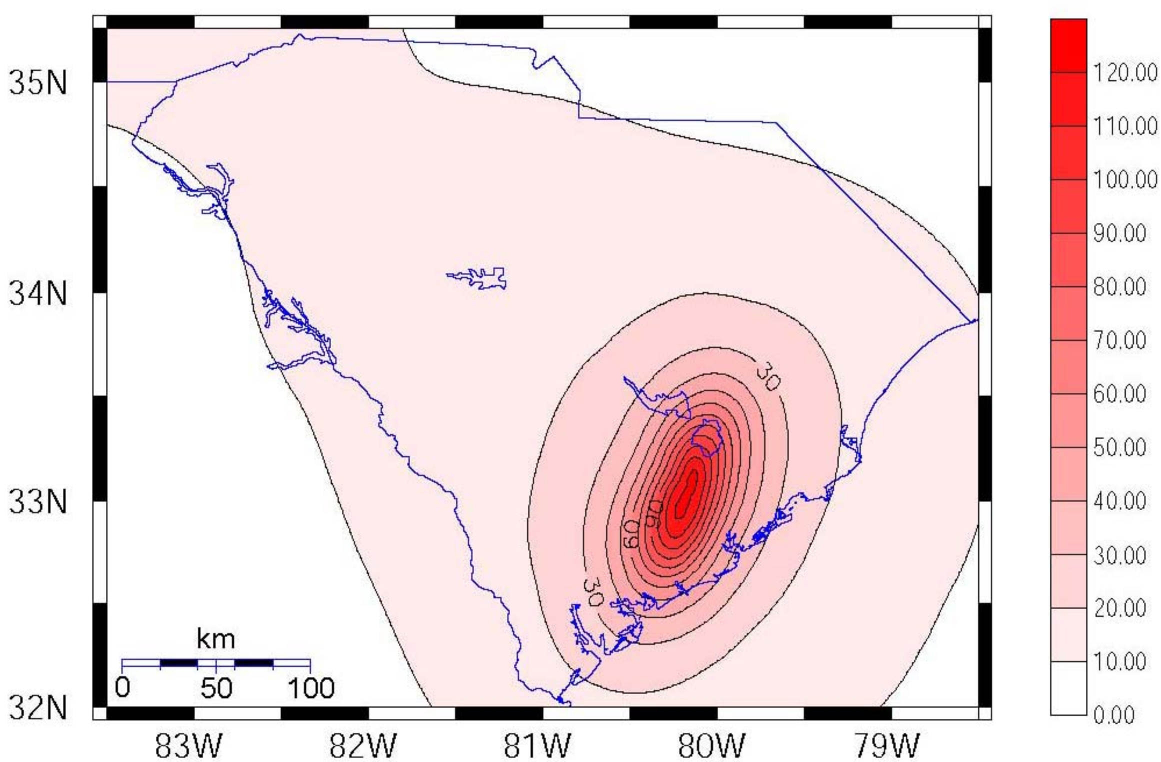


Figure 11-17, PGA (%g) - 3% P_E in 75 Years (2% P_E-50 Yr)
(Chapman and Talwani, 2002)

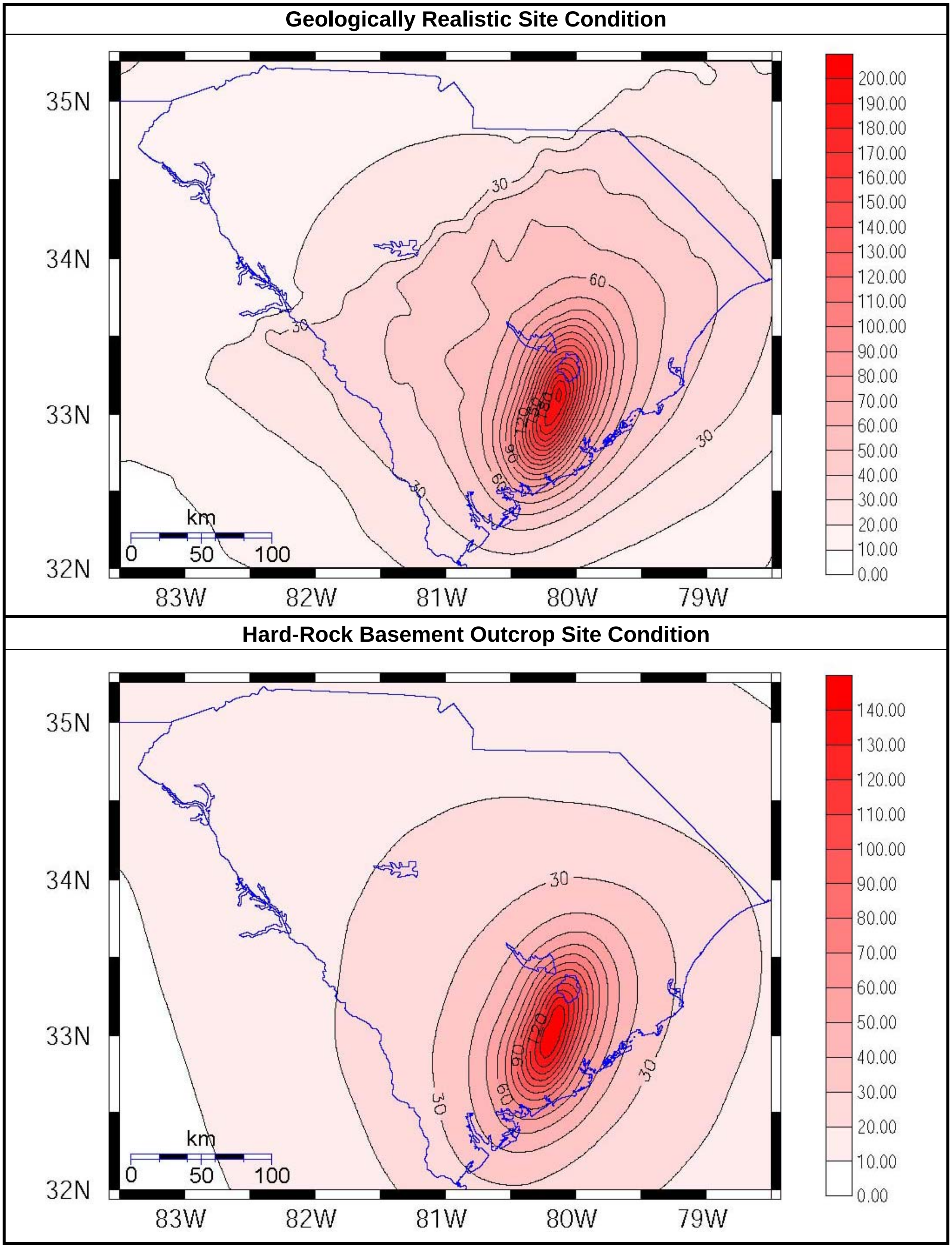
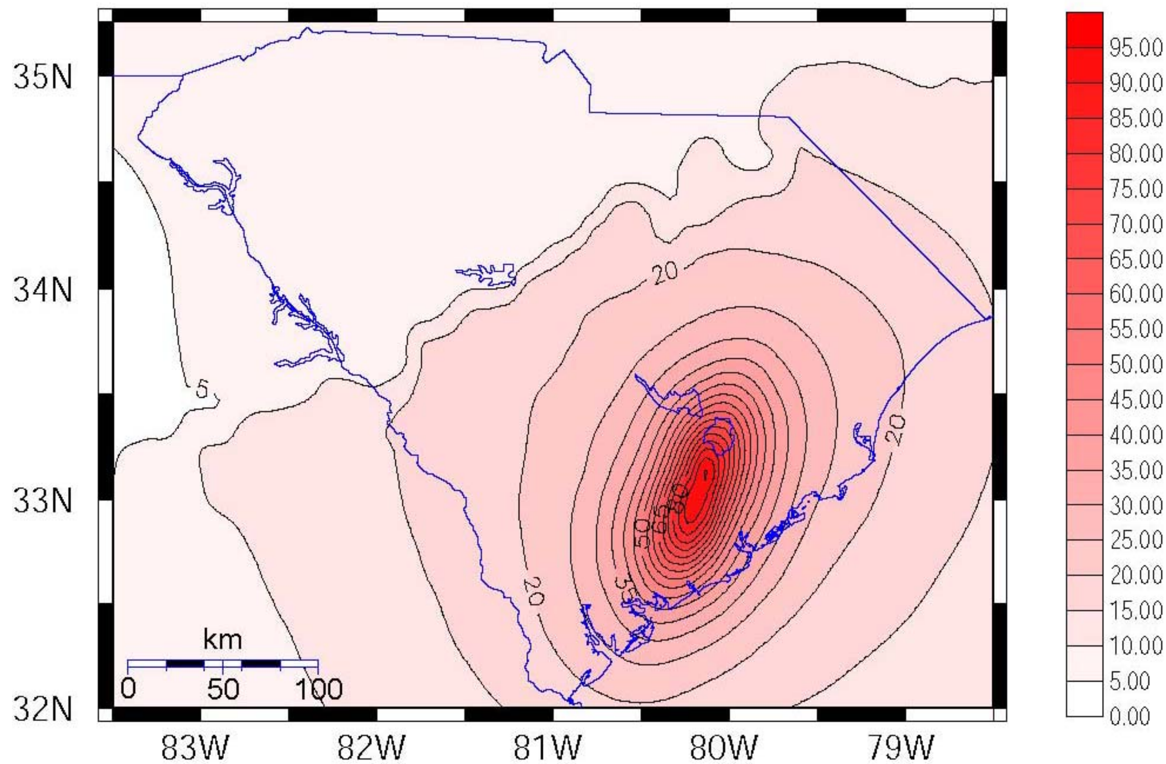


Figure 11-18, S_s Spectral Acceleration (%g) - 3% P_E in 75 Years (2% P_E-50 Yr) (Chapman and Talwani, 2002)

Geologically Realistic Site Condition



Hard-Rock Basement Outcrop Site Condition

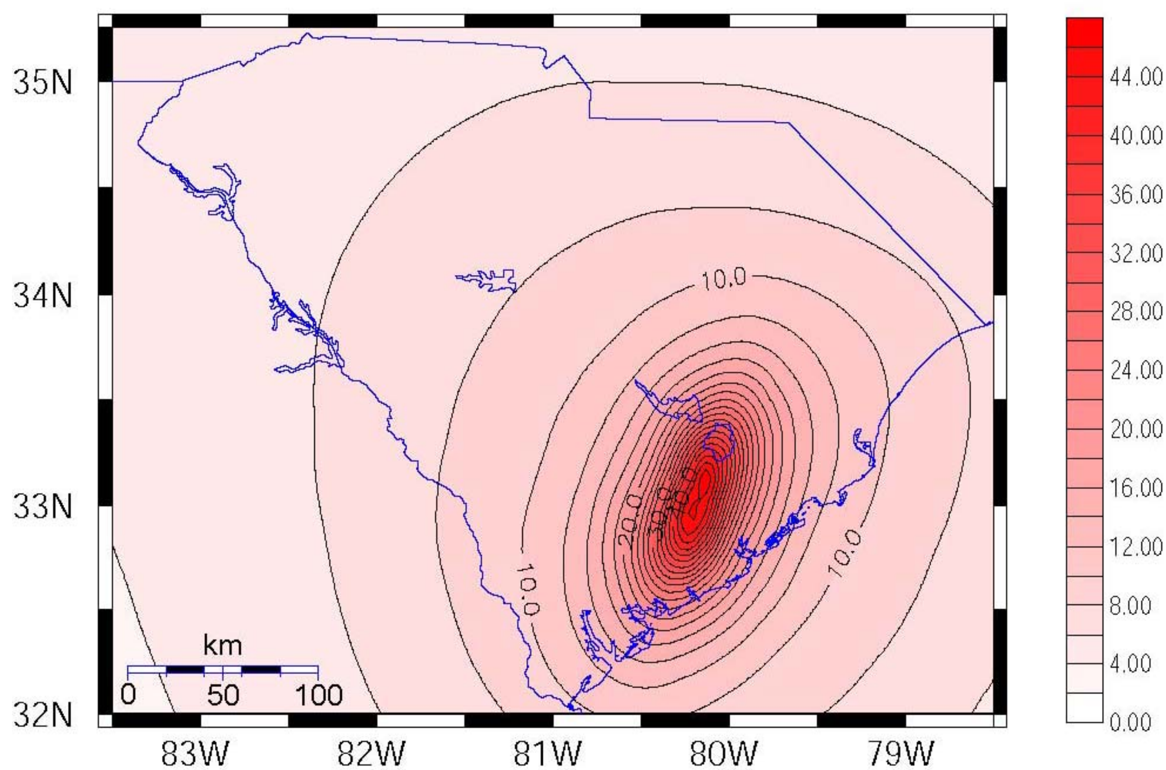


Figure 11-19, S_1 Spectral Acceleration (%g) - 3% P_E in 75 Years (2% P_E-50 Yr) (Chapman and Talwani, 2002)

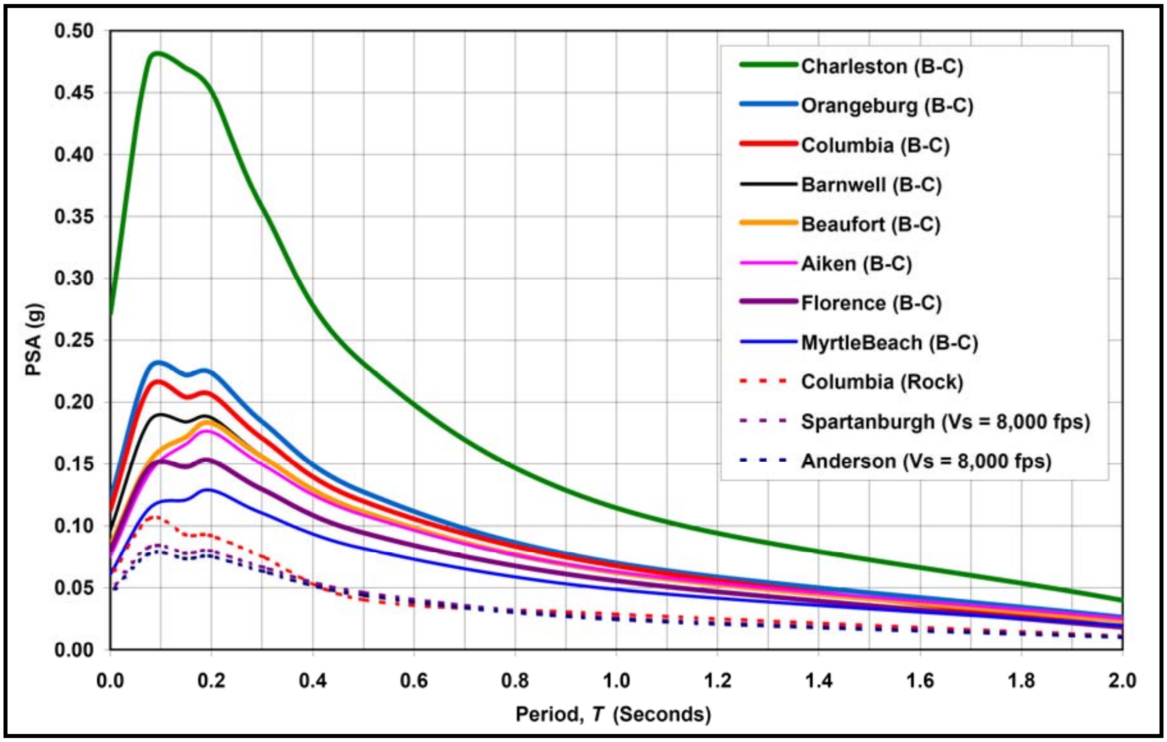


Figure 11-20, FEE PSA Curves for Selected South Carolina Cities

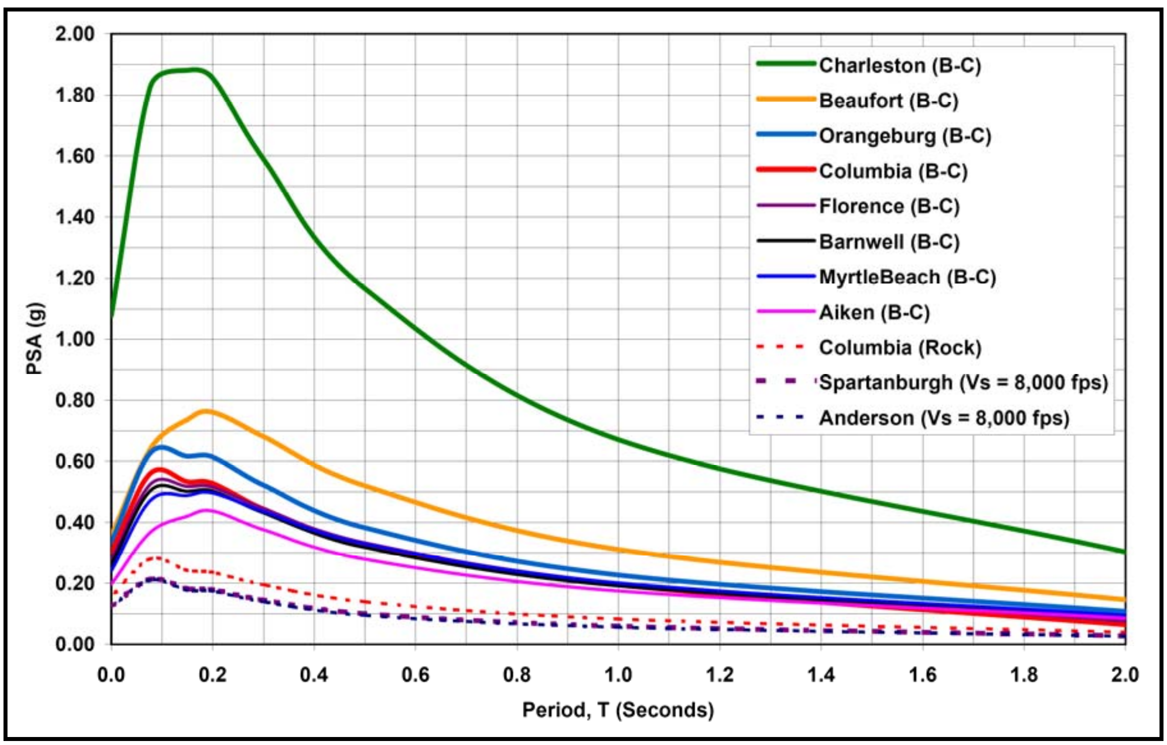


Figure 11-21, SEE PSA Curves for Selected South Carolina Cities

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)

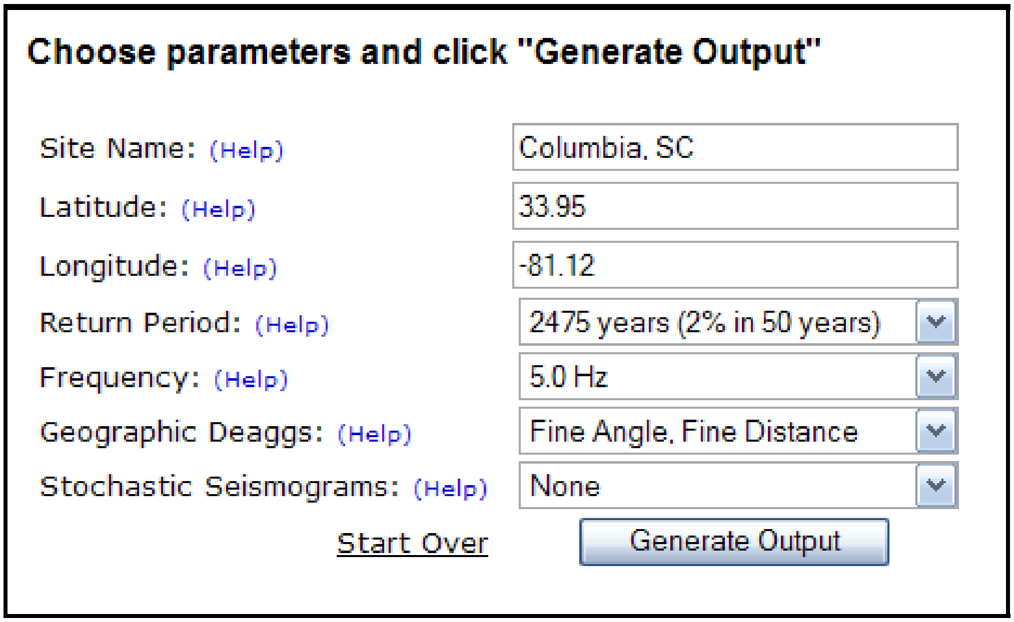


Figure 11-23, Interactive Deaggregation Input Screen (USGS 2002 Earthquake Deaggregations)

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.

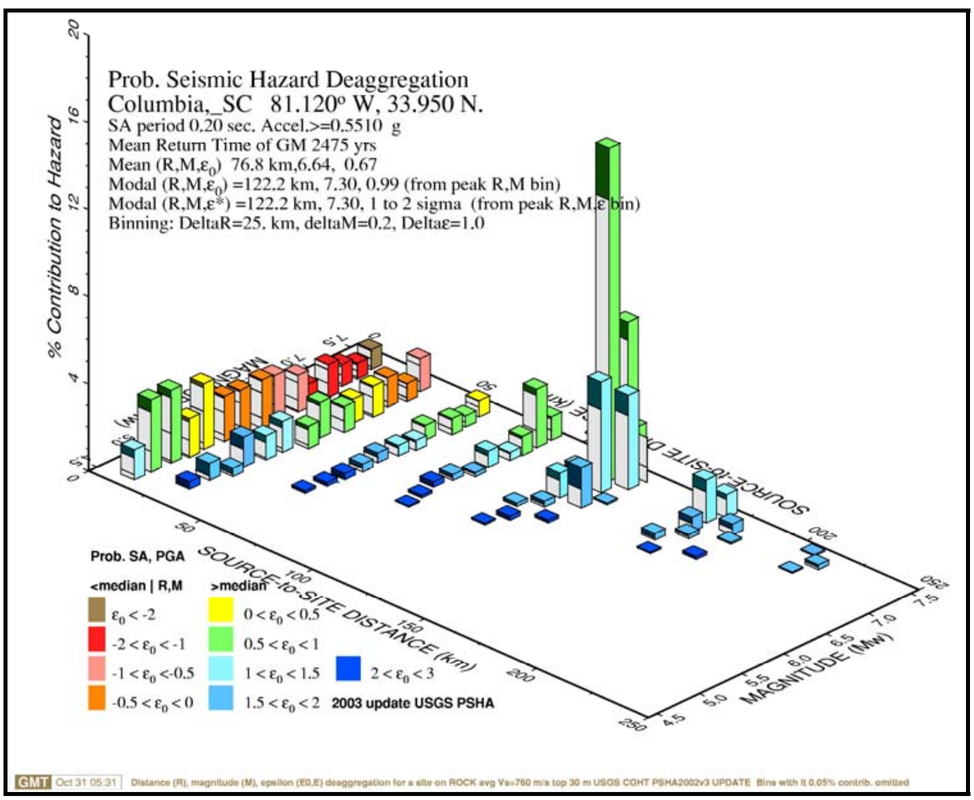


Figure 11-24, Columbia, SC Deaggregation SEE (3% P_E in 75 Years, 1Hz PSA) (USGS 2002 Earthquake Deaggregations)

```
*******************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)

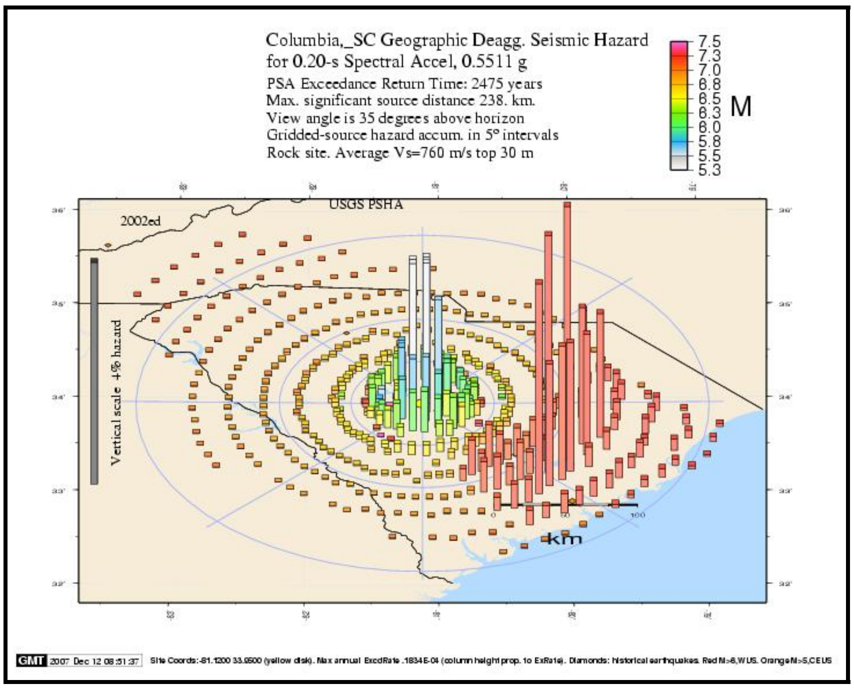


Figure 11-26, Geographic Deaggregation (Optional) (USGS 2002 Earthquake Deaggregations)

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 Ground Motions

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 (V_s = 8,200 ft/sec) equivalent to Site Class A (V_s > 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 (V_s > 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.

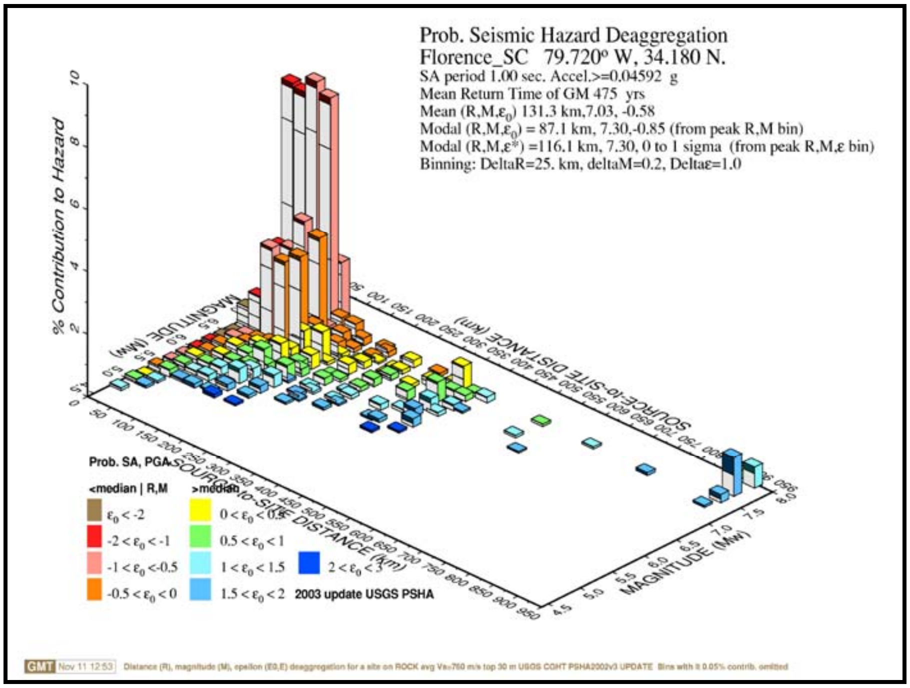


Figure 11-27, Florence, SC Deaggregation FEE (15% P_E in 75 Years, 1Hz PSA) (USGS 2002 Earthquake Deaggregations)

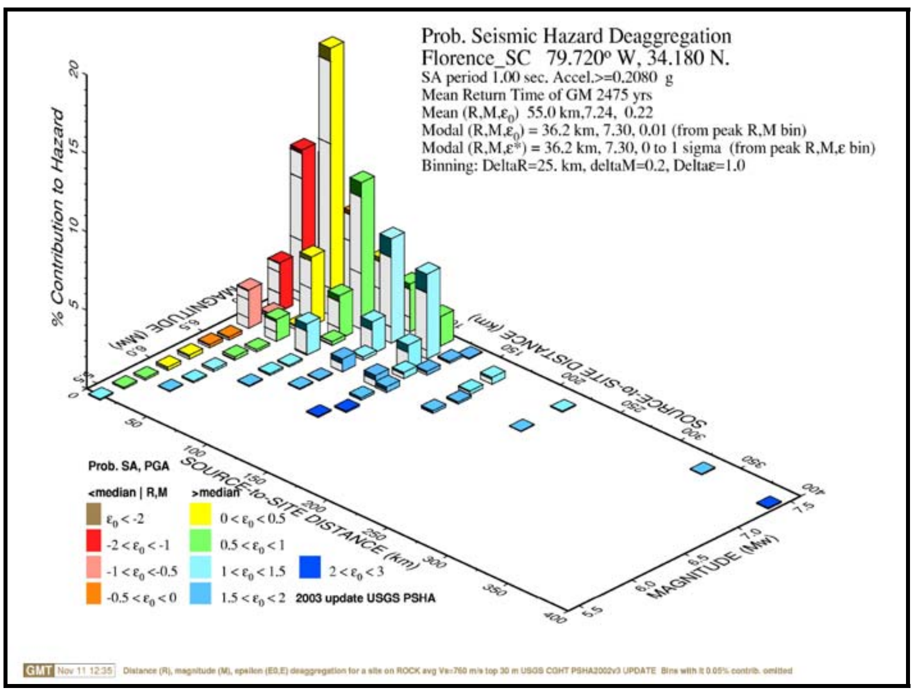


Figure 11-28, Florence, SC Deaggregation SEE (3% P_E in 75 Years, 1Hz PSA) (USGS 2002 Earthquake Deaggregations)

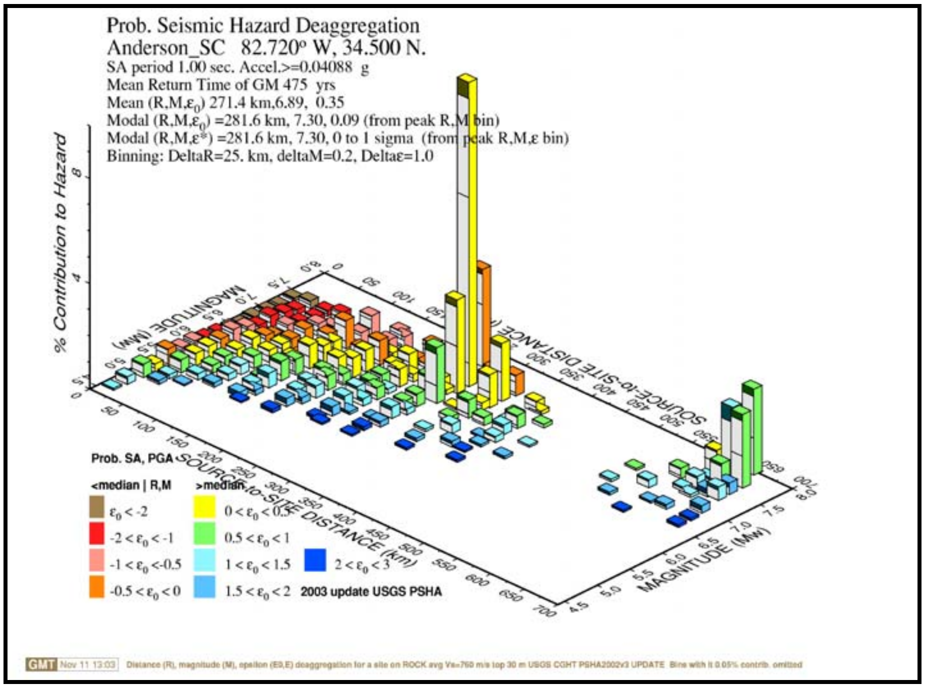


Figure 11-29, Anderson, SC Deaggregation FEE (15% P_E in 75 Years, 1Hz PSA) (USGS 2002 Earthquake Deaggregations)

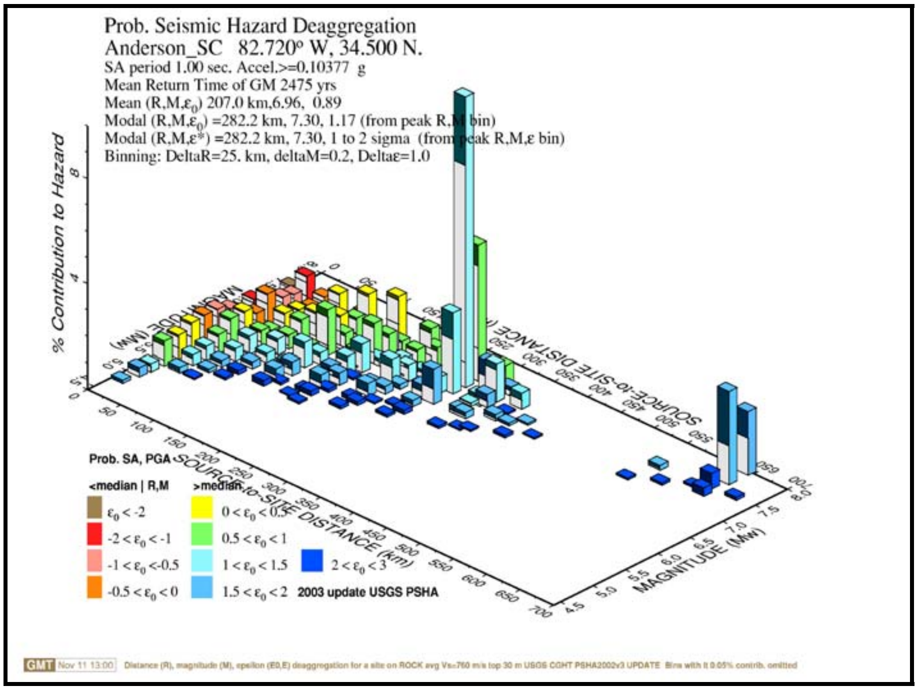


Figure 11-30, Anderson, SC Deaggregation SEE (3% P_E in 75 Years, 1Hz PSA) (USGS 2002 Earthquake Deaggregations)

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.

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Chapter 12

GEOTECHNICAL EARTHQUAKE ENGINEERING

FINAL

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

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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 Soil Properties

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 Soil Stiffness

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)

<table>
<tr><th rowspan="2">Soil Type</th><th>Mass Density, ρ</th><th>Total Unit Weight, γ</th><th colspan="2">Small-strain Shear Wave Velocity, V_S</th><th colspan="2">Initial Shear Modulus, G_{max}</th></tr>
<tr><th>kg/m³</th><th>pcf</th><th>m/s</th><th>ft/s</th><th>kPa</th><th>psi</th></tr>
<tr><td>Soft Clay</td><td>1,600</td><td>100</td><td>40 – 90</td><td>130 – 300</td><td>2,600 – 13,000</td><td>400 – 2,000</td></tr>
<tr><td>Stiff Clay</td><td>1,680</td><td>105</td><td>65 – 140</td><td>210 – 500</td><td>7,000 – 33,000</td><td>1000 – 5,700</td></tr>
<tr><td>Loose Sand</td><td>1,680</td><td>105</td><td>130 – 280</td><td>420 – 920</td><td>28,400 – 131,700</td><td>4,000 – 19,200</td></tr>
<tr><td>Dense Sand and Gravel</td><td>1,760</td><td>110</td><td>200 - 410</td><td>650 – 1,350</td><td>70,400 – 300,000</td><td>10,000 – 43,300</td></tr>
<tr><td>Residual Soil (PWR, IGM)</td><td>2,000</td><td>125</td><td>300 - 600</td><td>1,000 – 2,000</td><td>180,000 – 720,000</td><td>27,000 – 108,000</td></tr>
<tr><td>Piedmont Metamorphic and Igneous Rock (Highly – Moderately Weathered)</td><td rowspan="2">2,500</td><td rowspan="2">155</td><td>760 – 3,000</td><td>2,500 – 10,000</td><td rowspan="2">1,400,00 – 22,500,000</td><td rowspan="2">209,000 – 3,400,000</td></tr>
<tr><td>0 <RQD < 50
RQD = 65 [(1)]
RQD = 80 [(1)]
RQD = 90 [(1)]
RQD = 100 [(1)]</td><td>600
760
1,500
2,500
3,400</td><td>2,000
2,500
5,000
8,000
11,000</td></tr>
<tr><td>Basement Rock (Moderately Weathered to Intact)</td><td>2,600</td><td>165</td><td>> 3,400</td><td>> 11,000</td><td>> 30,000</td><td>> 4,300,000</td></tr>
</table>

[(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%	$V_S = 72.9(N^*_{60})^{0.224} Z^{0.130} ASF$	Equation 12-2
10% to 35%	$V_S = 72.3(N^*_{60})^{0.228} Z^{0.152} ASF$	Equation 12-3
< 10%	$V_S = 66.7(N^*_{60})^{0.248} Z^{0.138} ASF$	Equation 12-4

[1] N^*_{60} = blows/0.3m = blows/ft (Section 7.8.1) and Z = depth in meters, ASF = Age Scaling Factors

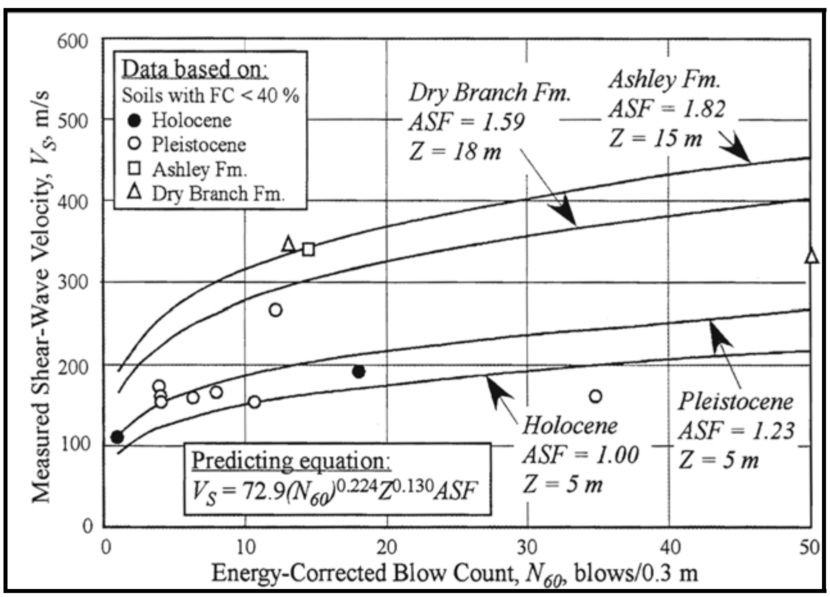


Figure 12-1, SPT (N_{60}) vs. Shear Wave (V_S) (Andrus et al., 2003)

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

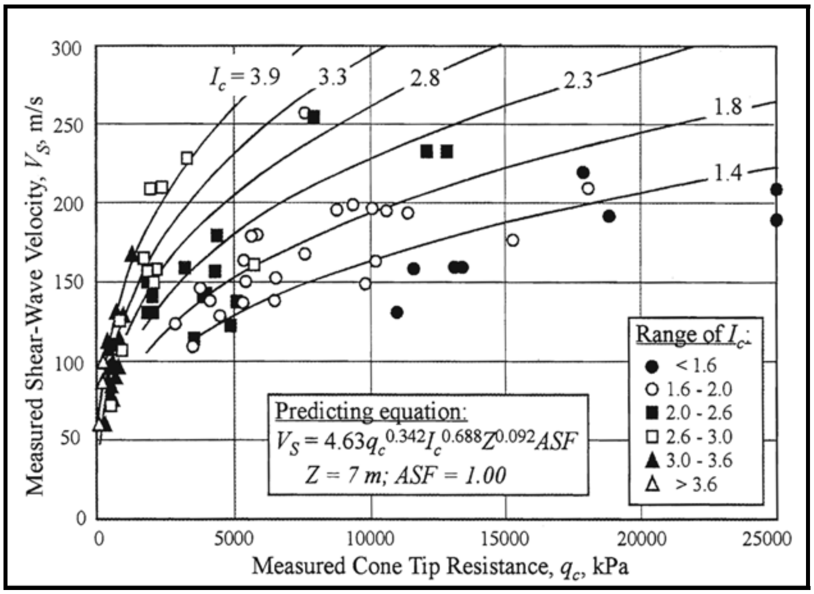


Figure 12-2, CPT – V_S Relationship for Holocene Soils (Andrus et al., 2003)

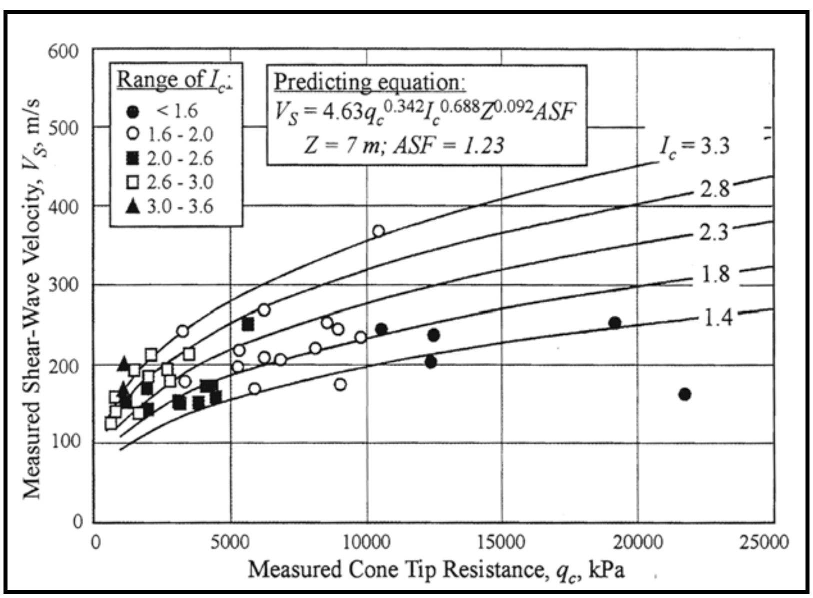


Figure 12-3, CPT – V_S Relationship for Pleistocene Soils (Andrus et al., 2003)

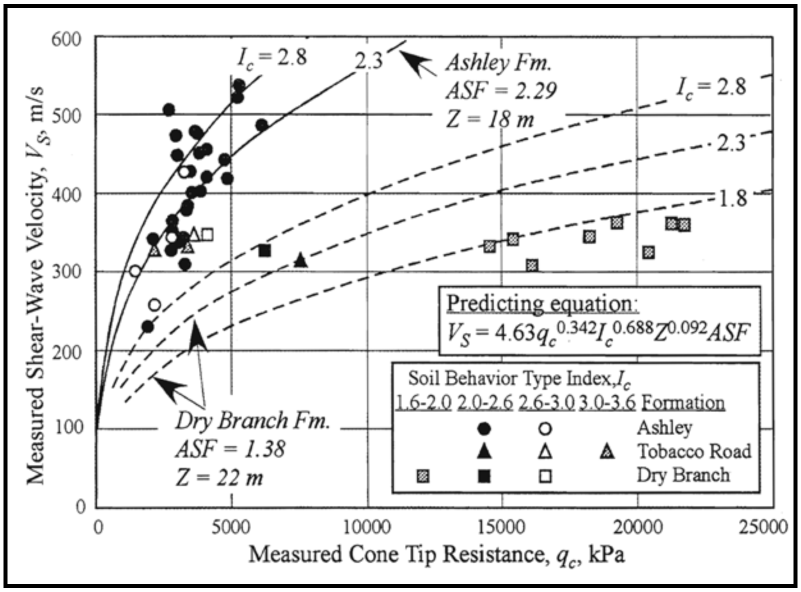


Figure 12-4, CPT – V_S Relationship for Tertiary Soils (Andrus et al., 2003)

Robertson (1990) established general soil behavior type, I_c, values as shown in Figure 12-5.

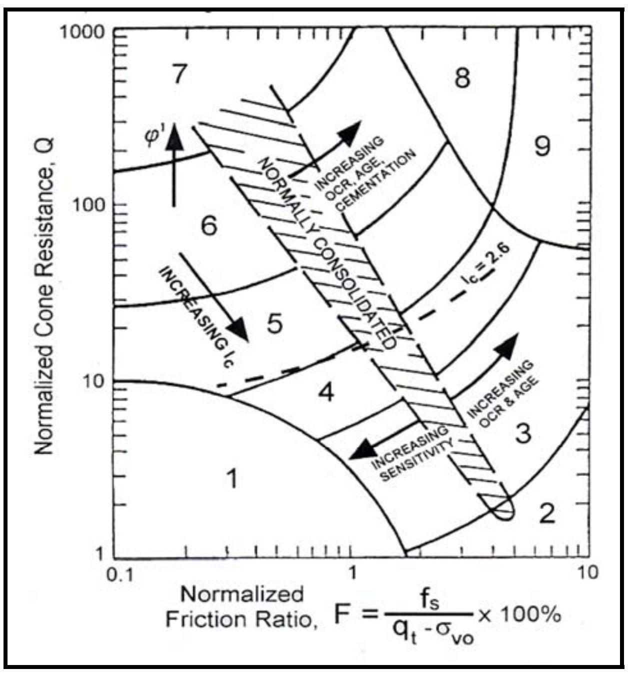


Figure 12-5, Normalized CPT Soil Behavior Type Chart (Robertson, 1990)

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.

$$V_{1,S} = V_S C_{VS} = V_S \left(\frac{P_a}{\sigma'_V} \right)^{0.25}$$ Equation 12-11

$$C_{VS} = \left(\frac{P_a}{\sigma'_V} \right)^{0.25} \leq 1.4$$ 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)

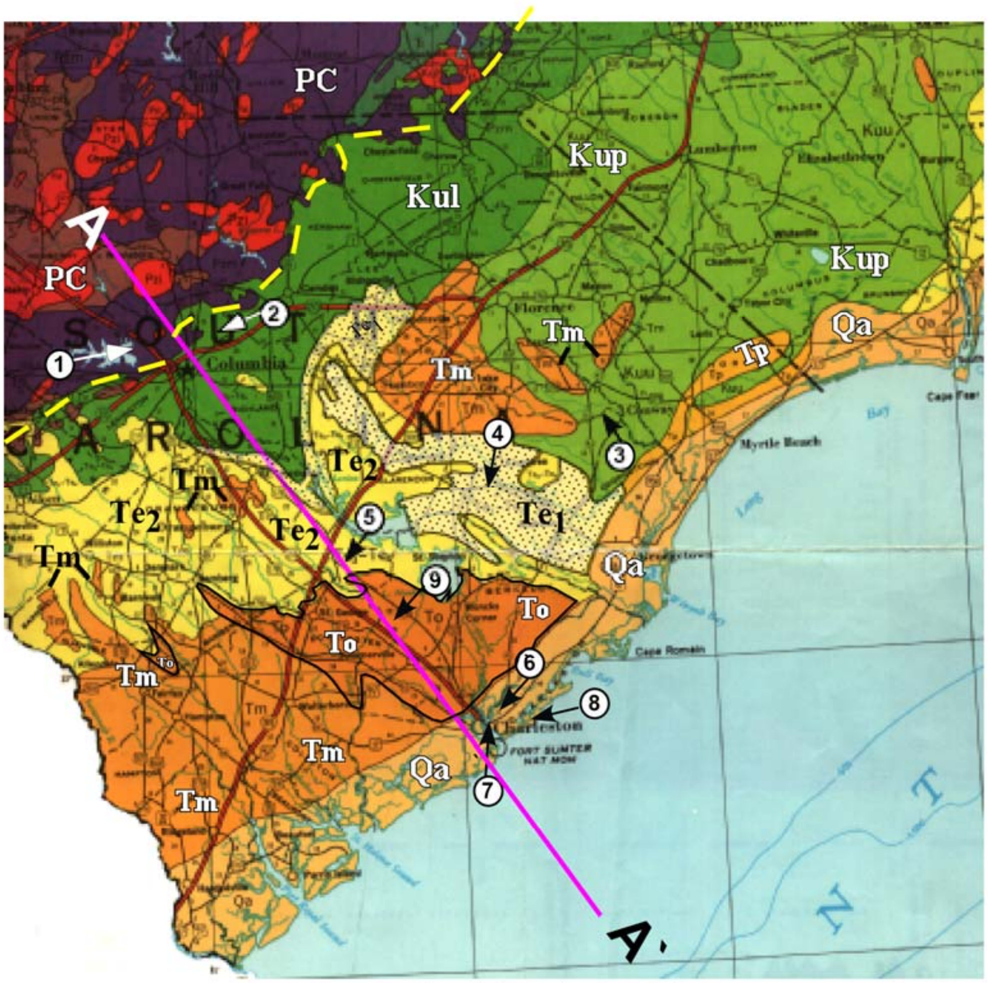
5. Santee Limestone: Santee Limestone (Middle Eocene-Clayborne Group)

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)	(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

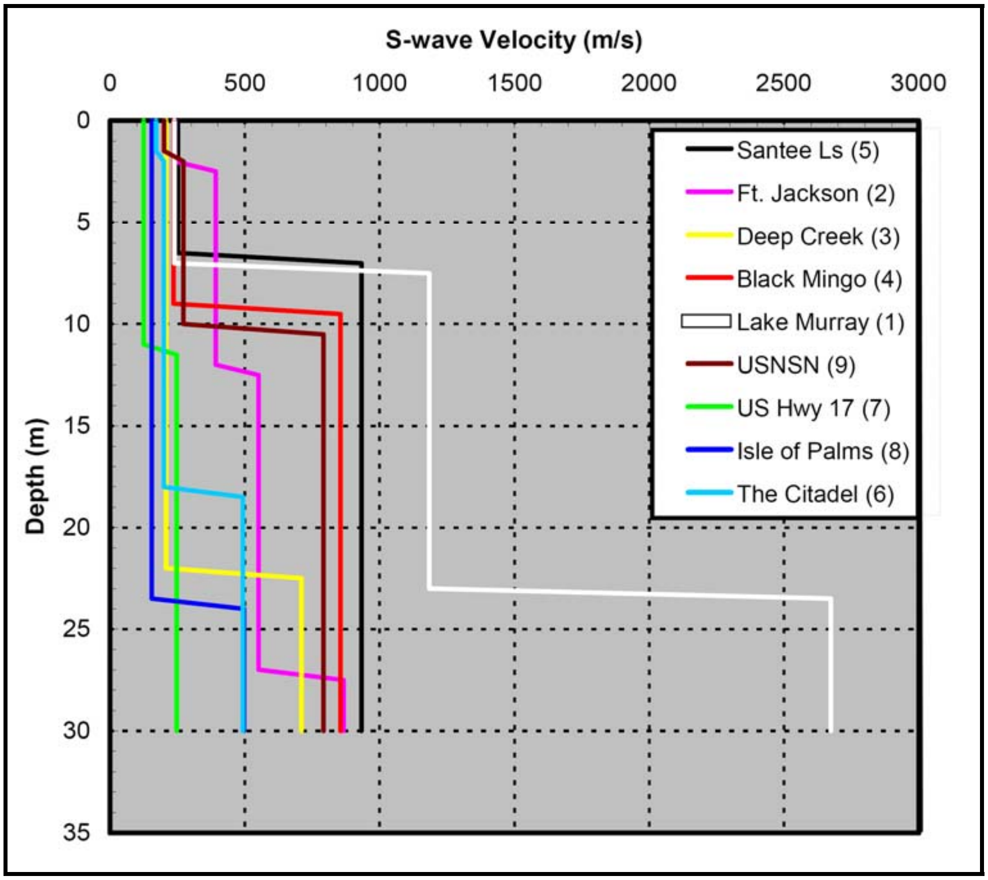


Figure 12-7, USGS Shear Wave V_S Profile (Odum et al., 2003)

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).

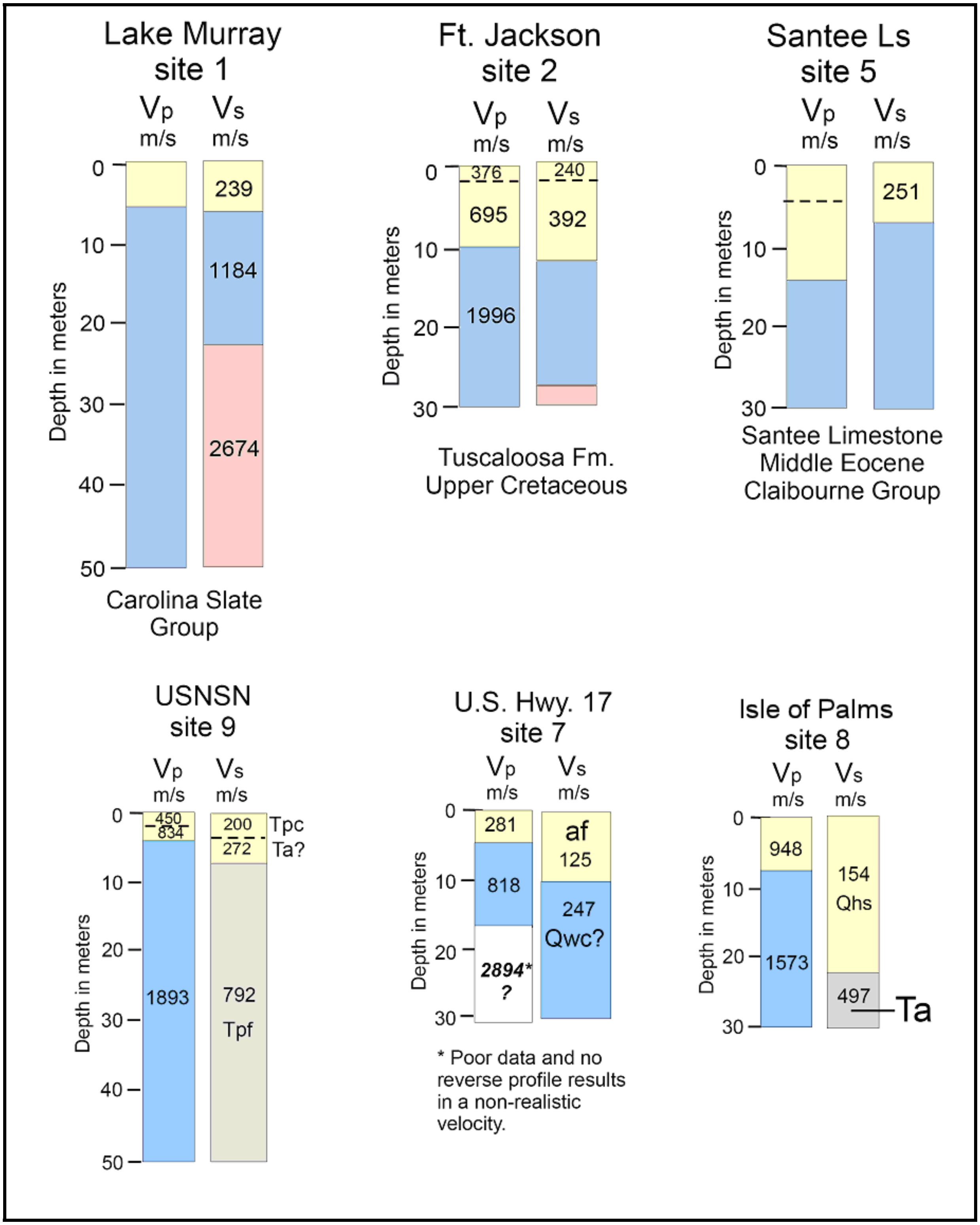


Figure 12-8, USGS Sites 1, 2, 5, 9, 7, and 8
(Odum et al., 2003)

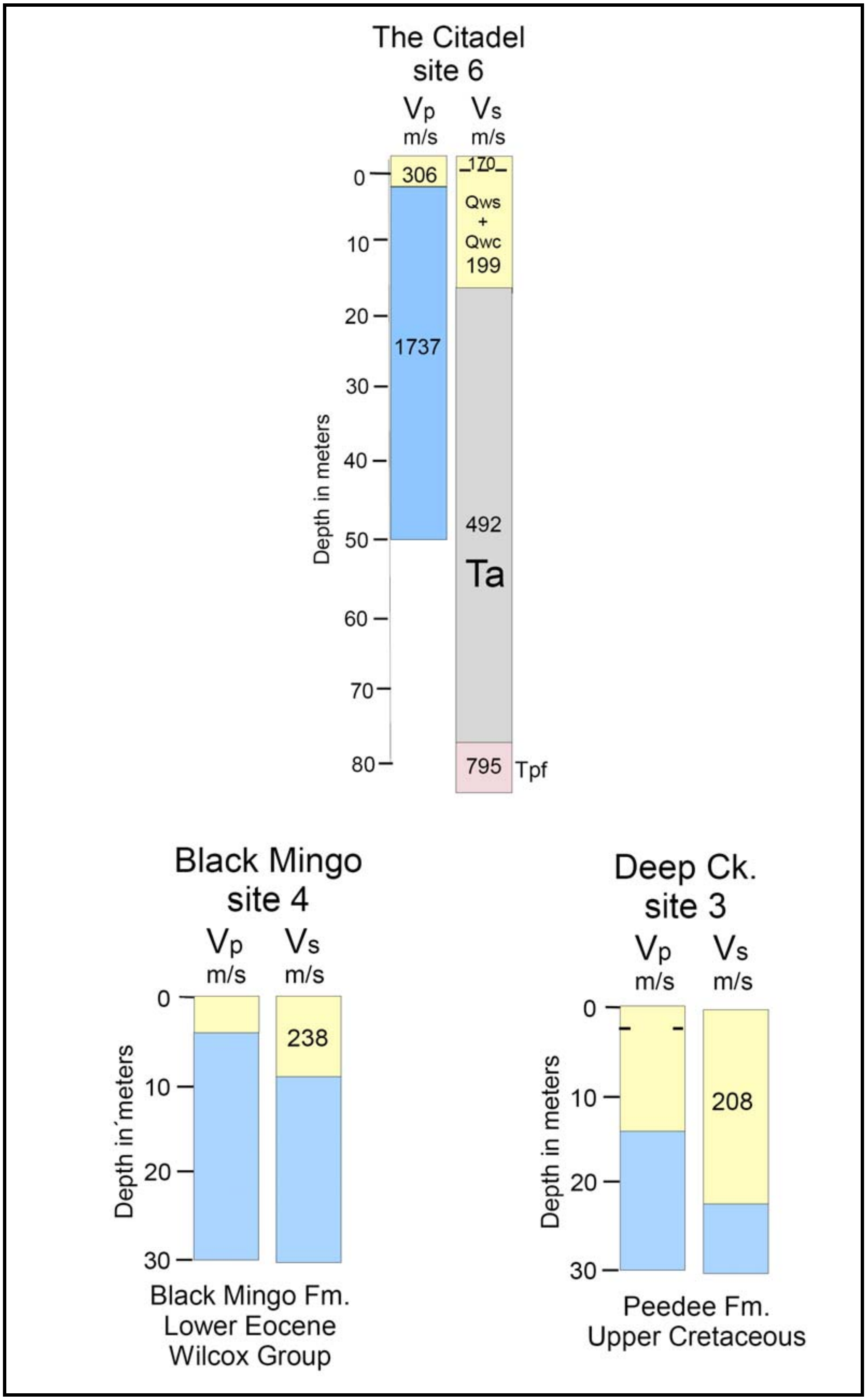


Figure 12-9, USGS Sites 6, 4, 3
(Odum et al., 2003)

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).

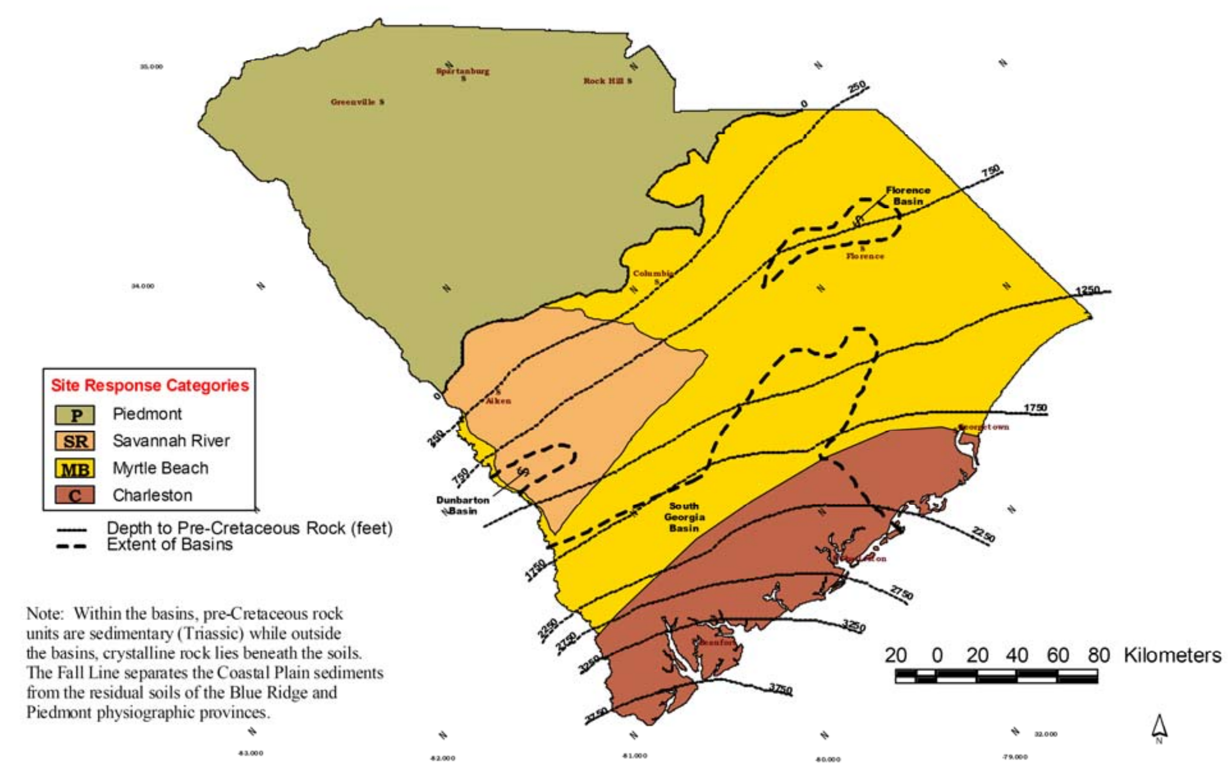


Figure 3-5. Site Response categories and depth to pre-Cretaceous rock.

Figure 12-10, Site Response Categories and Depth To Pre-Cretaceous Rock (URS Corporation, 2001)

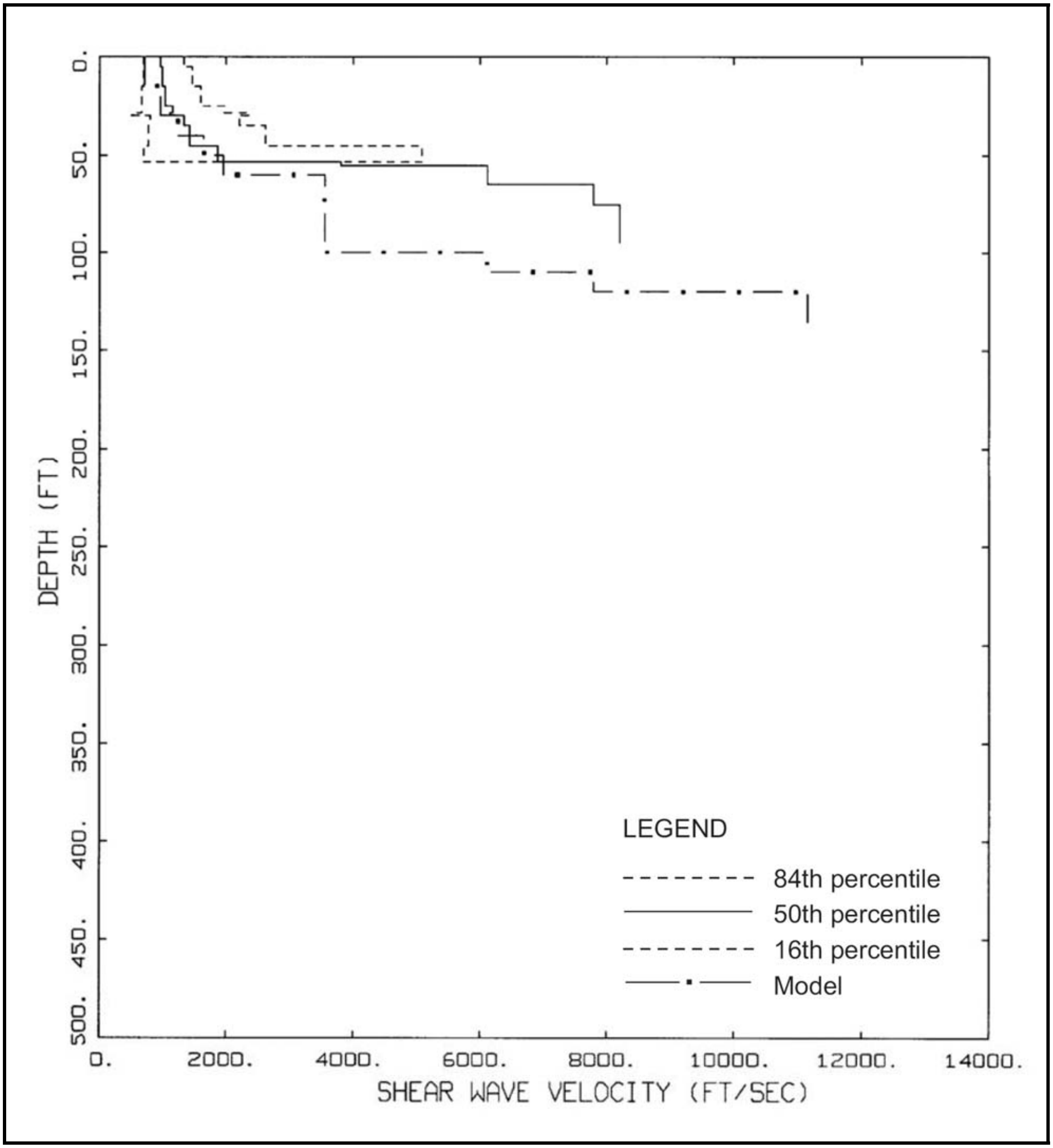


Figure 12-11, Piedmont/Blue Ridge Site Response Category Base Vs Profile (URS Corporation, 2001)

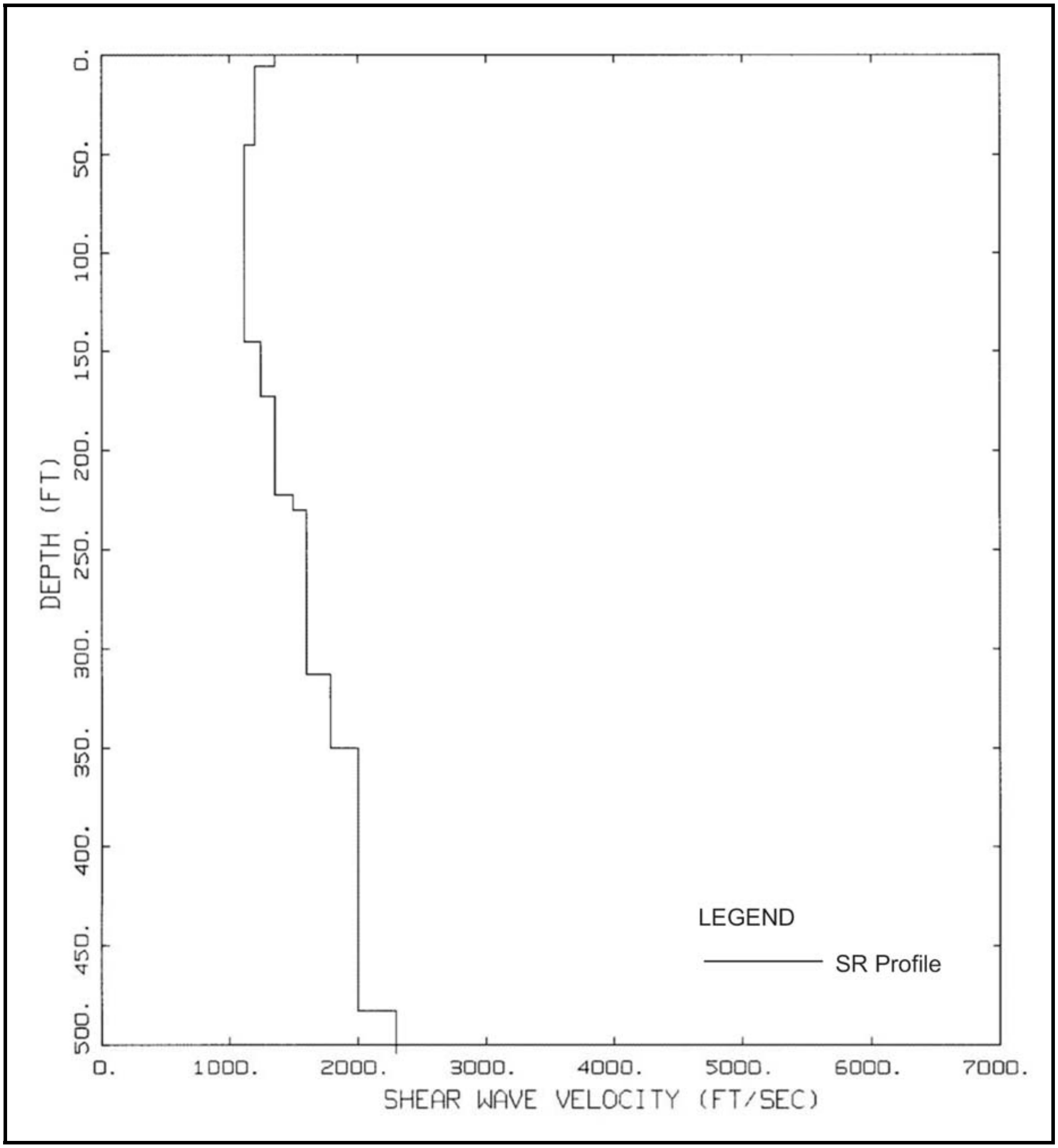


Figure 12-12, Savannah River Site Response Category Base Vs Profile (URS Corporation, 2001)

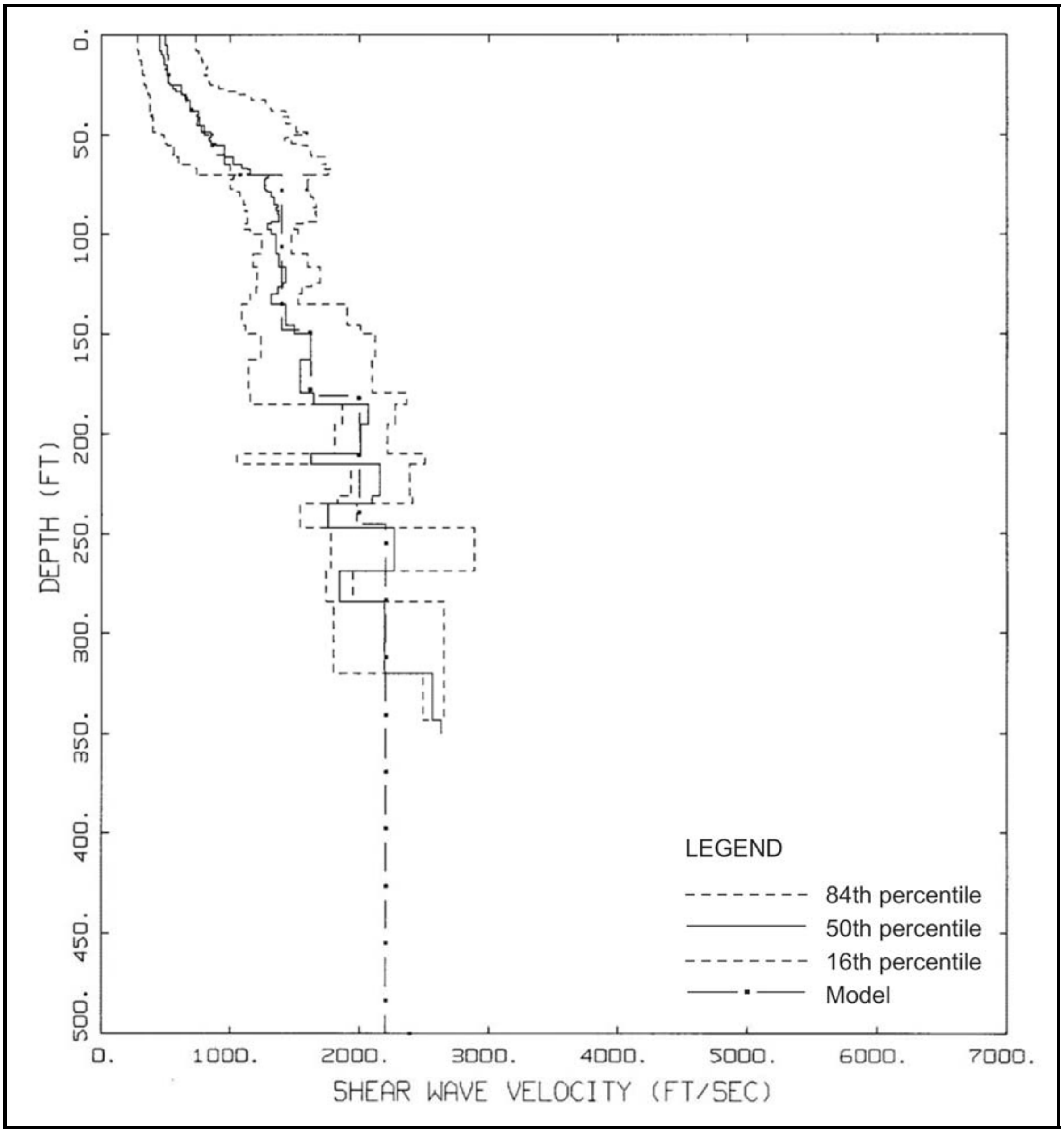


Figure 12-13, Charleston Site Response Category Base Vs Profile (URS Corporation, 2001)

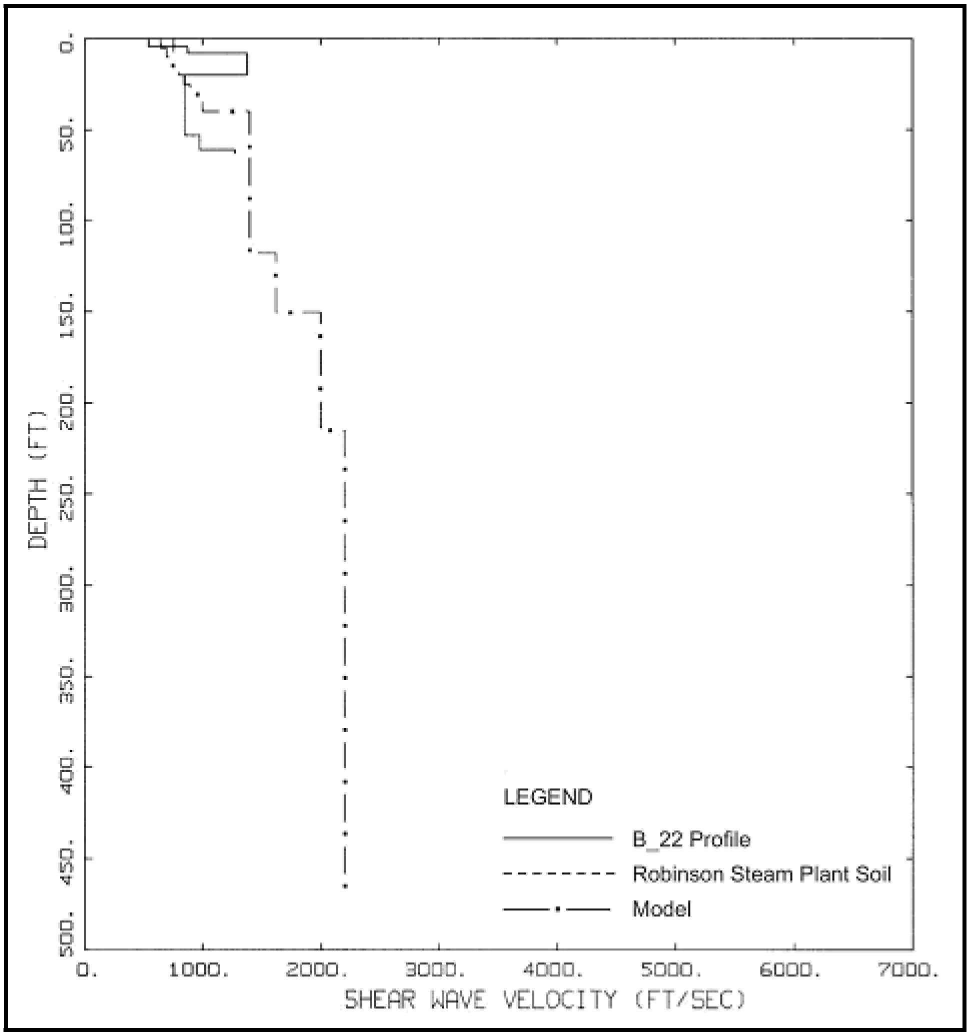


Figure 12-14, Myrtle Beach Site Response Category Base Vs Profile (URS Corporation, 2001)

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.

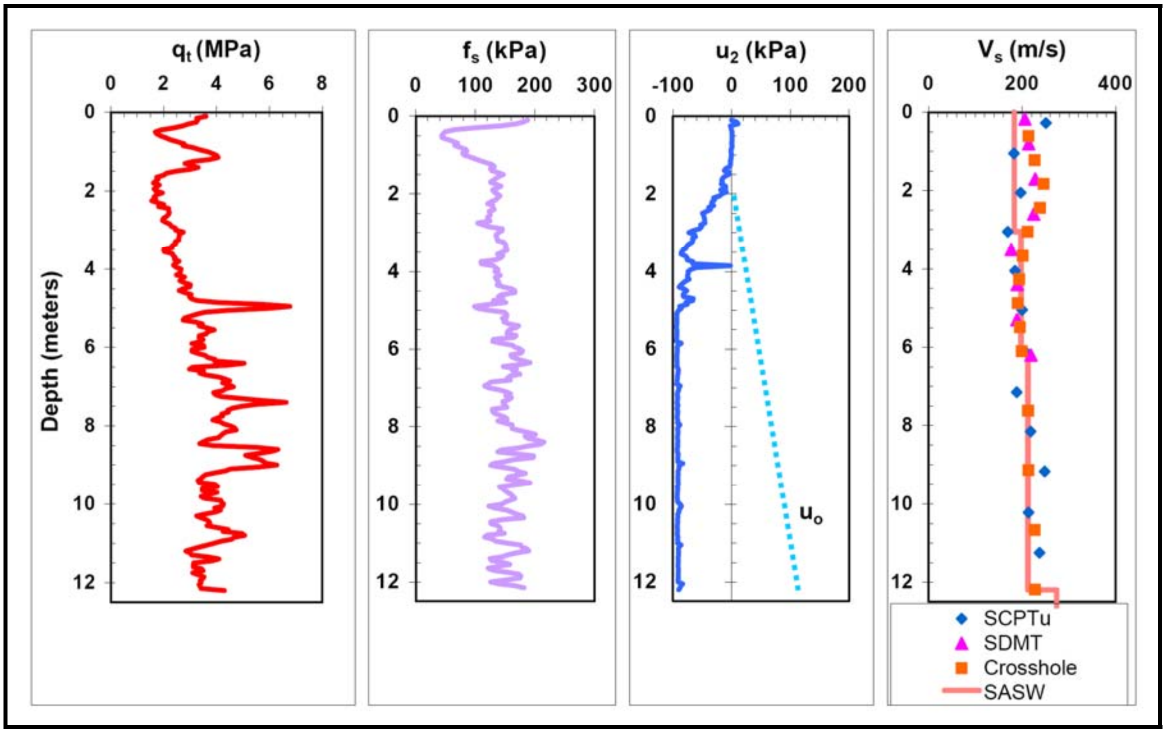


Figure 12-15, SCPT Piedmont Profile - NGES Opelika, Alabama (Mayne et al., 2000)

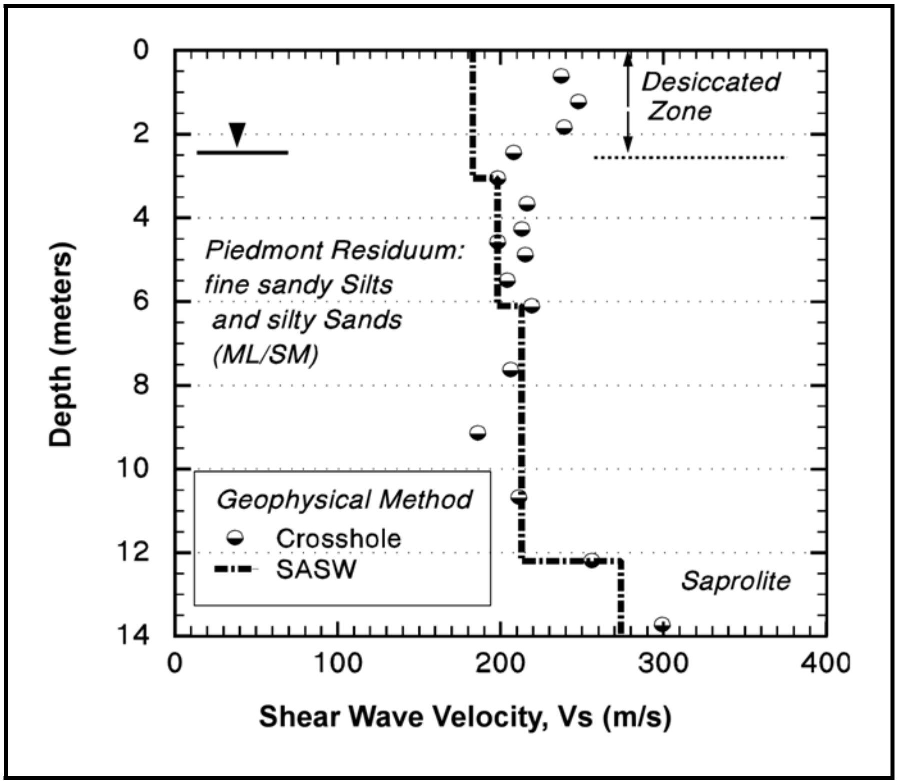


Figure 12-16, Geophysical V_S Piedmont Profile - NGES Opelika, Alabama (Mayne et al., 2000)

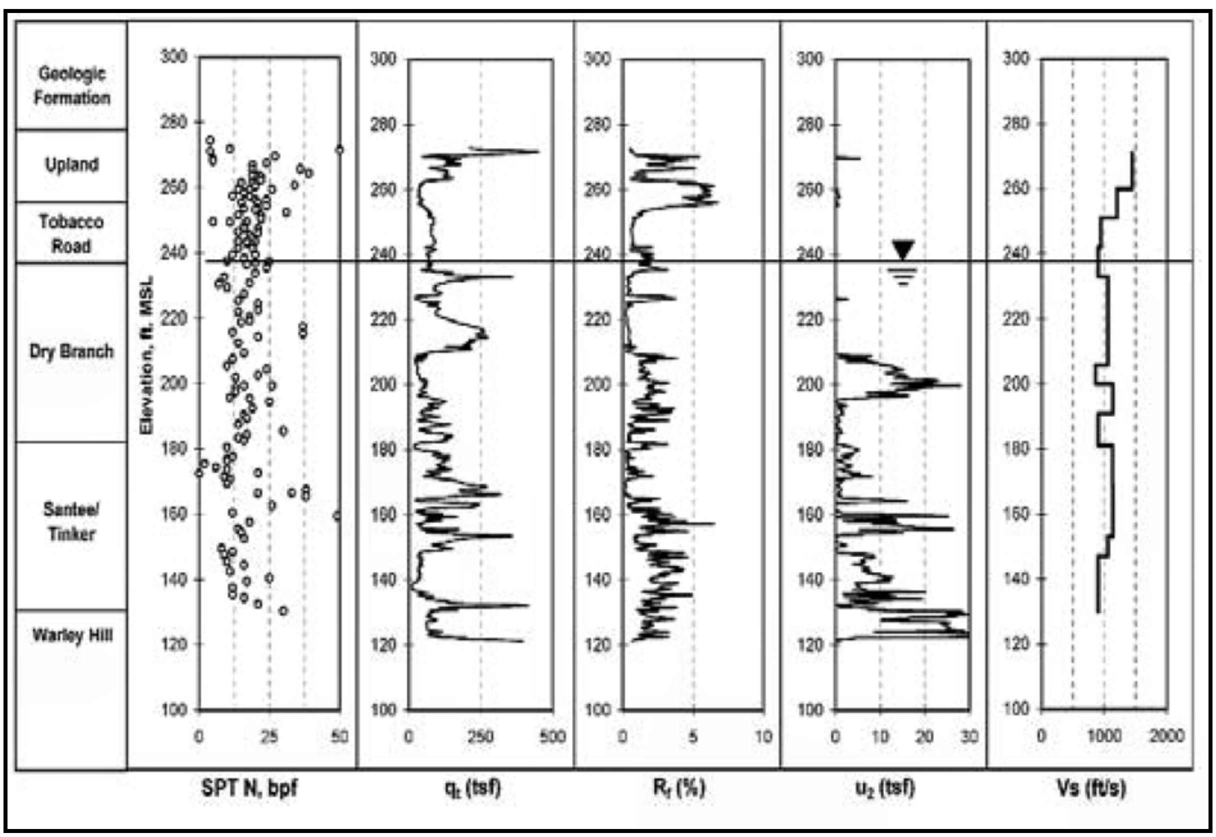


Figure 12-17, SCPT Profile Savannah River, South Carolina (Lewis et al., 2004)

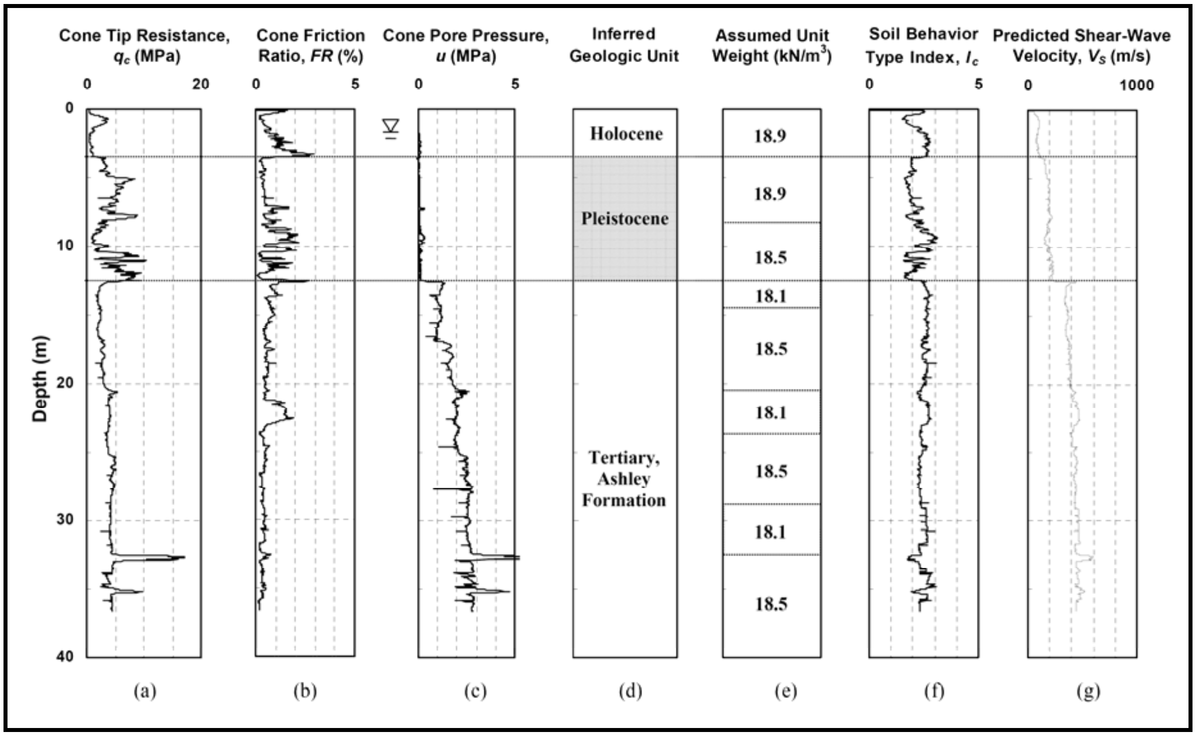


Figure 12-18, SCPT Profile (DS-1) Cooper River Bridge, Charleston, SC (S&ME, 2000)

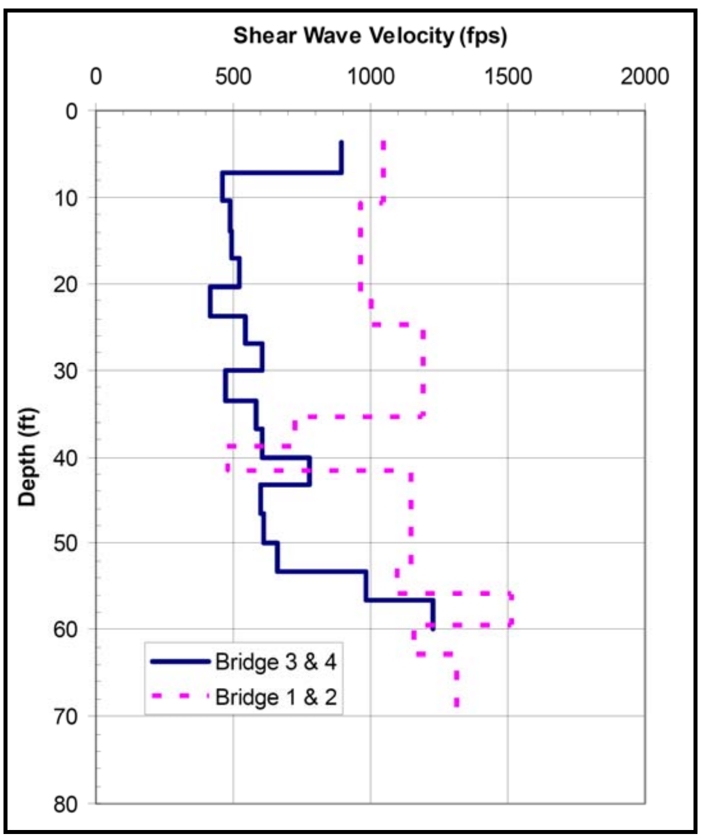


Figure 12-19, Shear Wave Profile US 17, Beaufort County, South Carolina (S&ME, 2007)

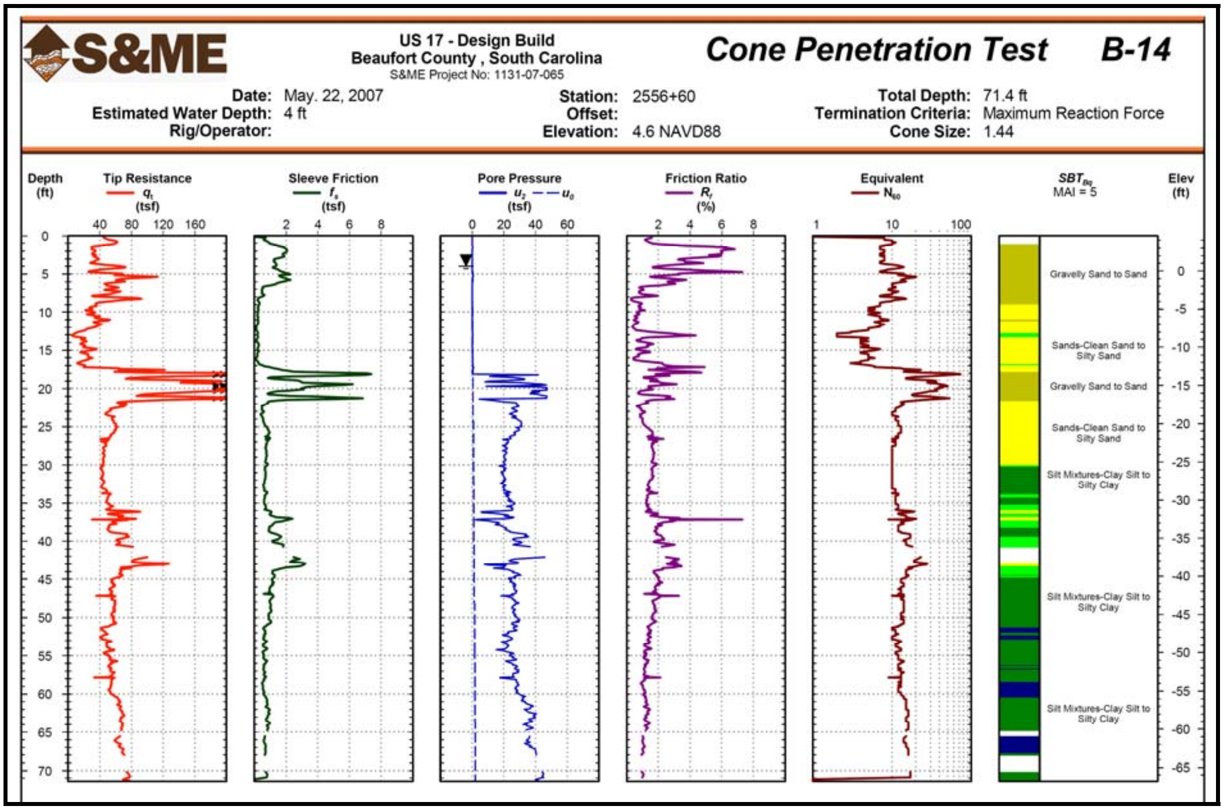


Figure 12-20, SCPT (B-14) US 17 Bridge 1, Beaufort County, South Carolina (S&ME, 2007)

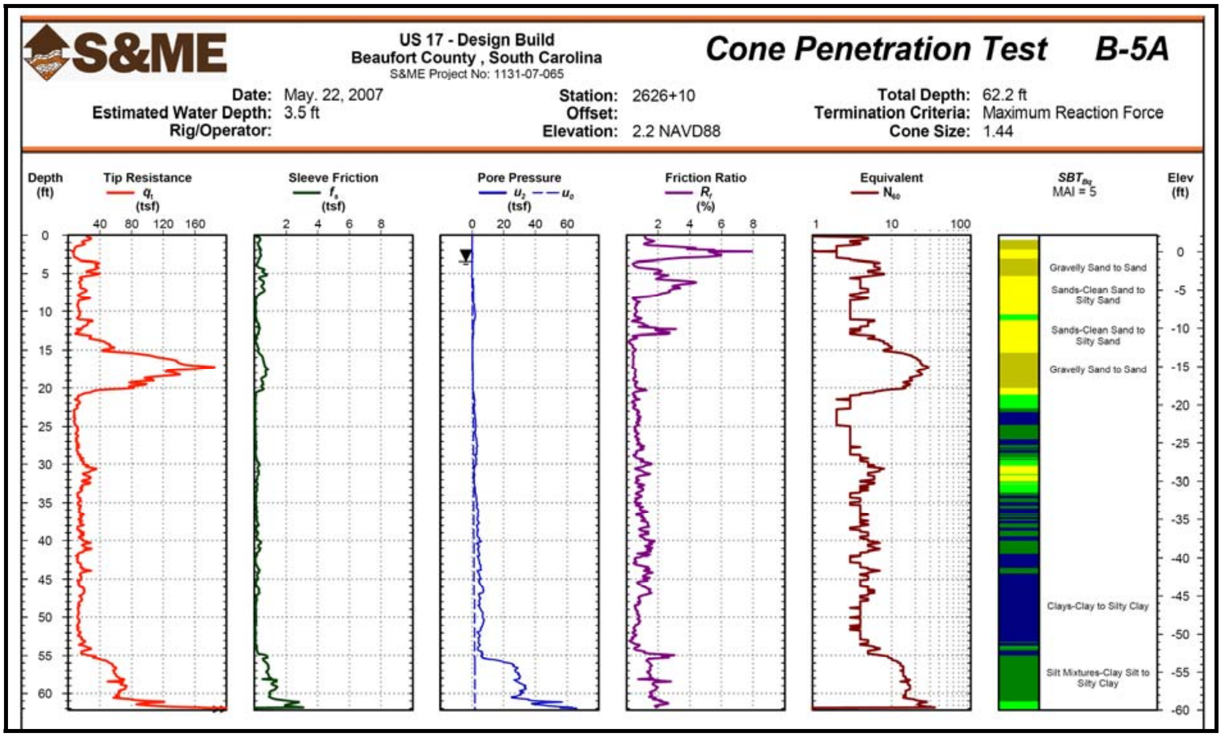
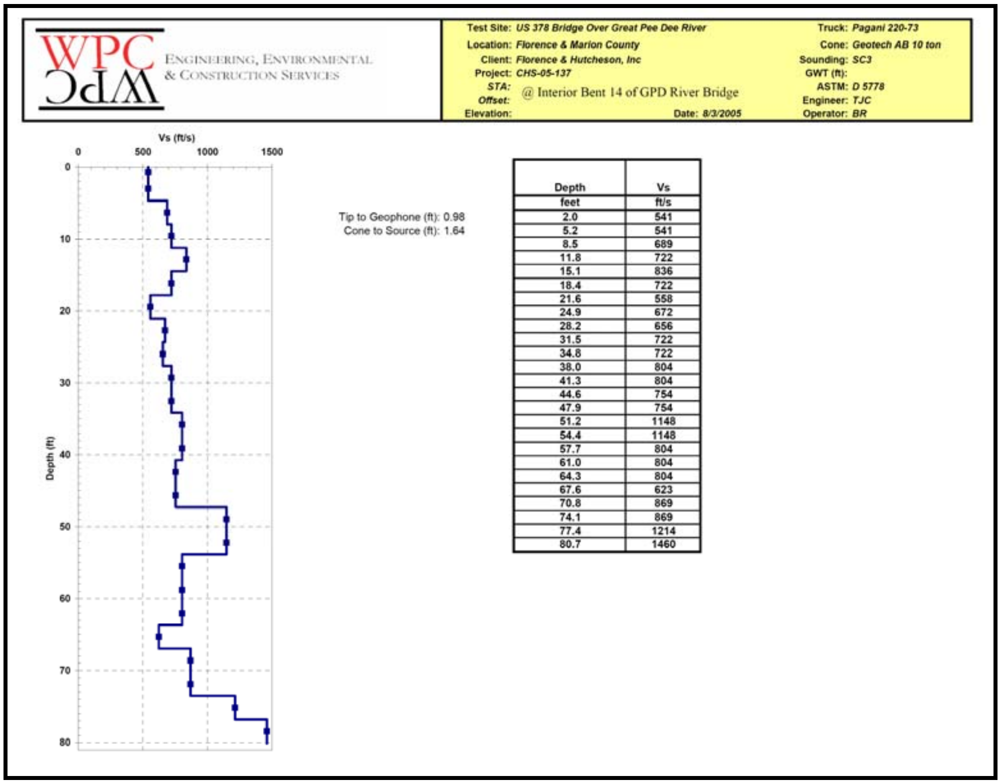


Figure 12-21, SCPT (B-5A) US 17 Bridge 3, Beaufort County, South Carolina (S&ME, 2007)



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)

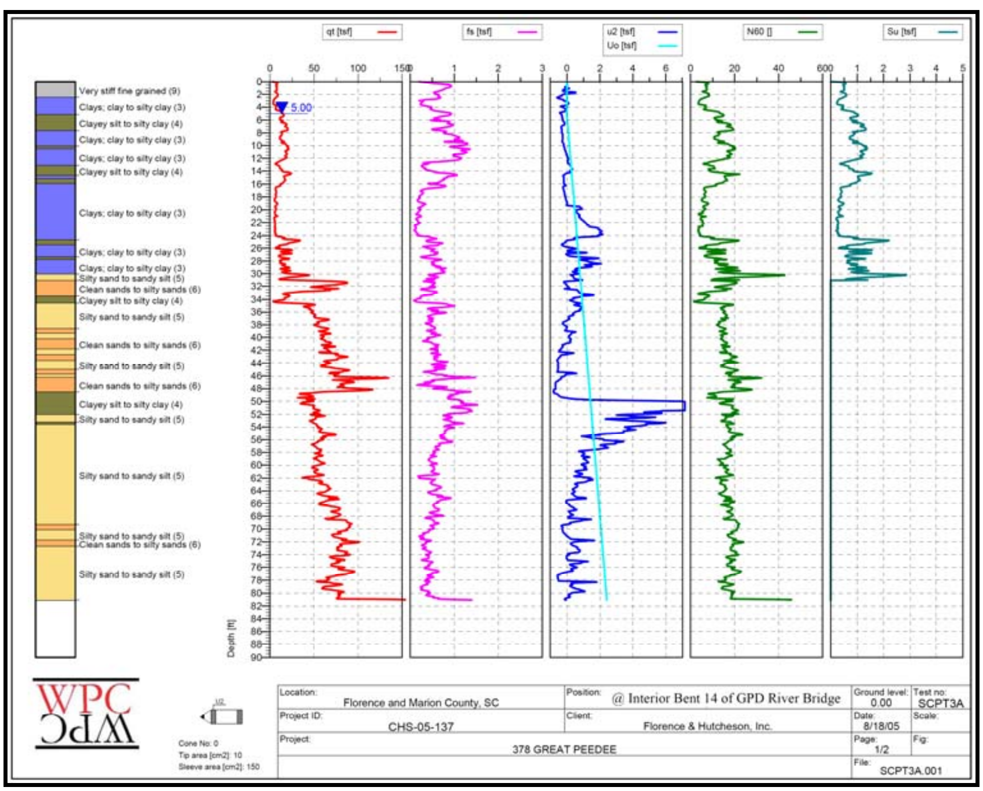
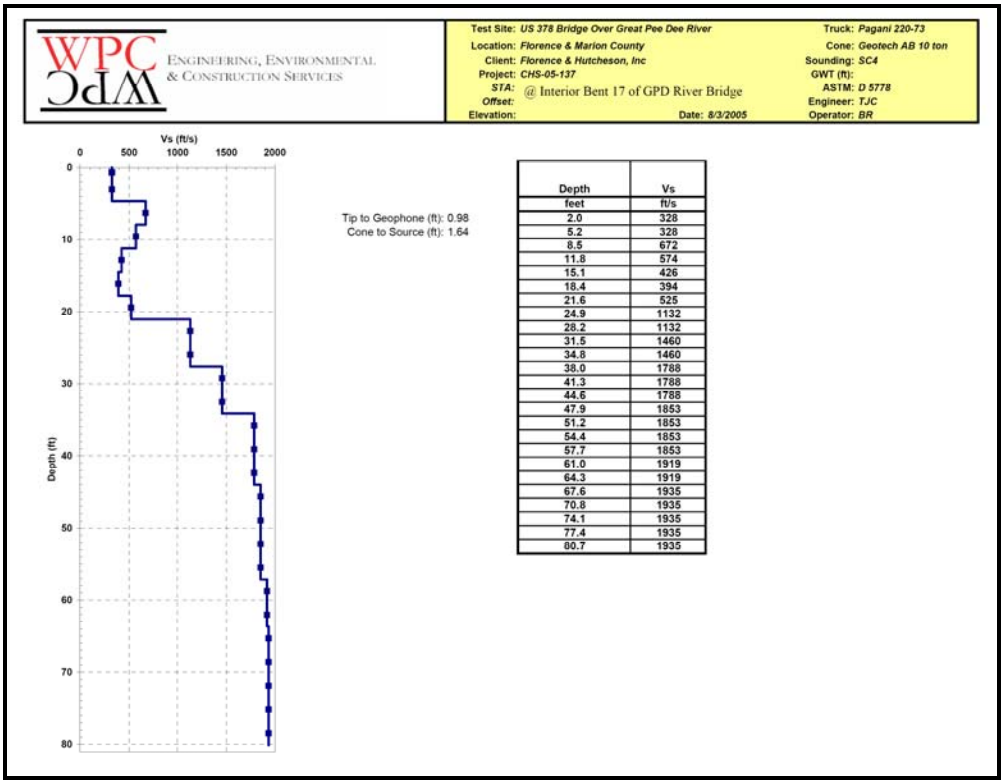


Figure 12-23, SCPT (SC3) - US 378, Lake City, South Carolina (Florence & Hutcheson, 2006)



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)

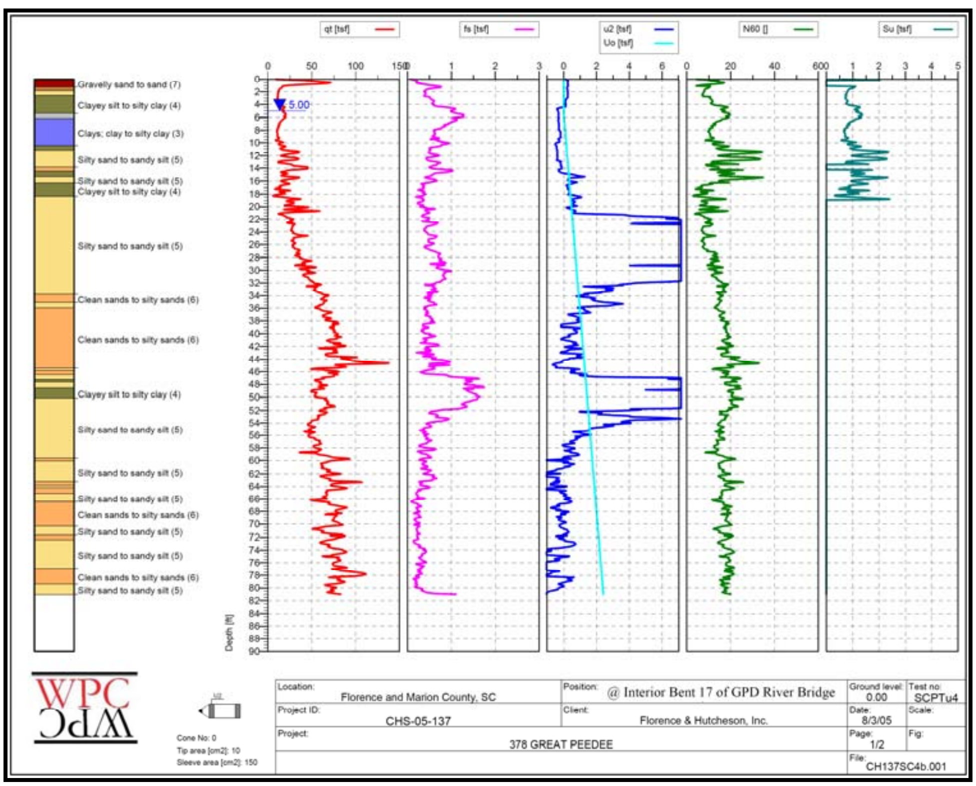


Figure 12-25, SCPT (SC4) - US 378, Lake City, South Carolina (Florence & Hutcheson, 2006)

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)

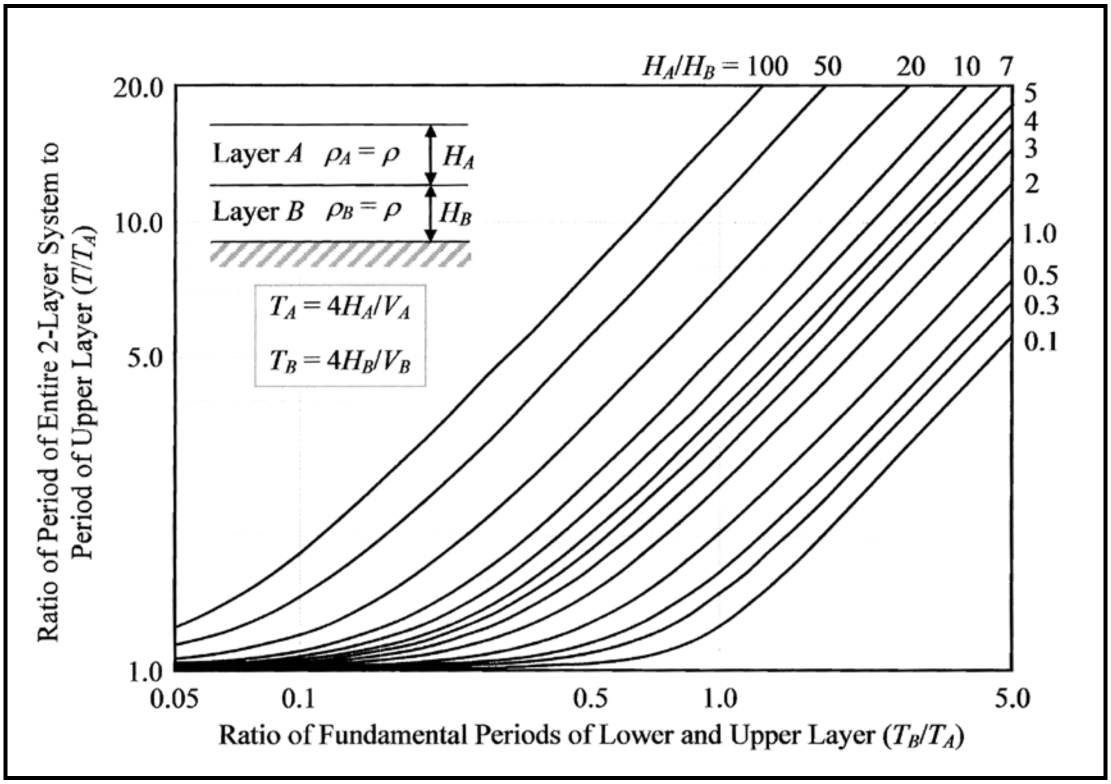


Figure 12-26, Fundamental Period of Two-Layer System (Oweis et al., 1975, Adapted by Green (2001))

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.

$$\sigma'_m = \sigma'_v \left[\frac{1 + 2K'_o}{3} \right] \qquad \text{Equation 12-21}$$

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 <u>geologic unit</u> using Equation 12-21
4	Calculate σ'_m for each <u>layer</u> 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 <u>layer</u> 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 <u>geologic unit</u> (or "subdivided" geologic unit) that has a corresponding average σ'_m.
7	Compute the design shear modulus reduction curves (G/G_{max}) for each <u>layer</u> 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.

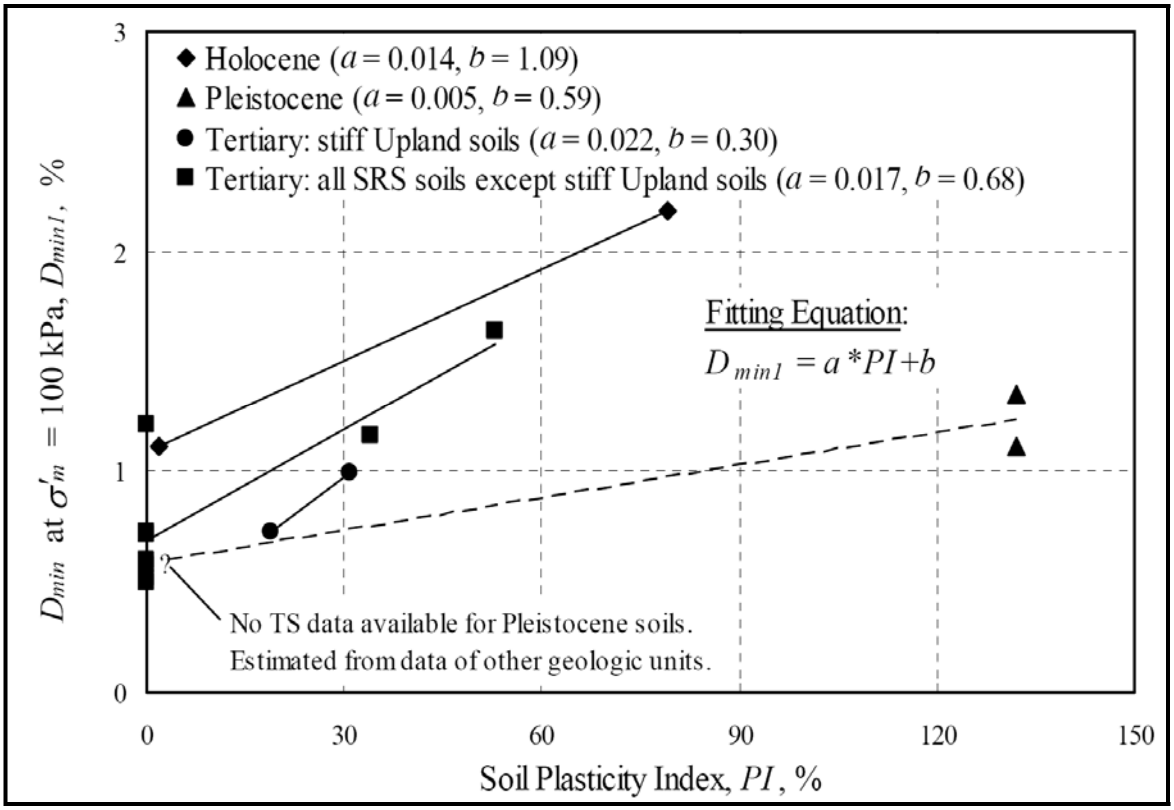


Figure 12-27, D_{min1}, Small-Strain Damping @ σ'_m = 1 atm (Andrus et al., 2003)

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.

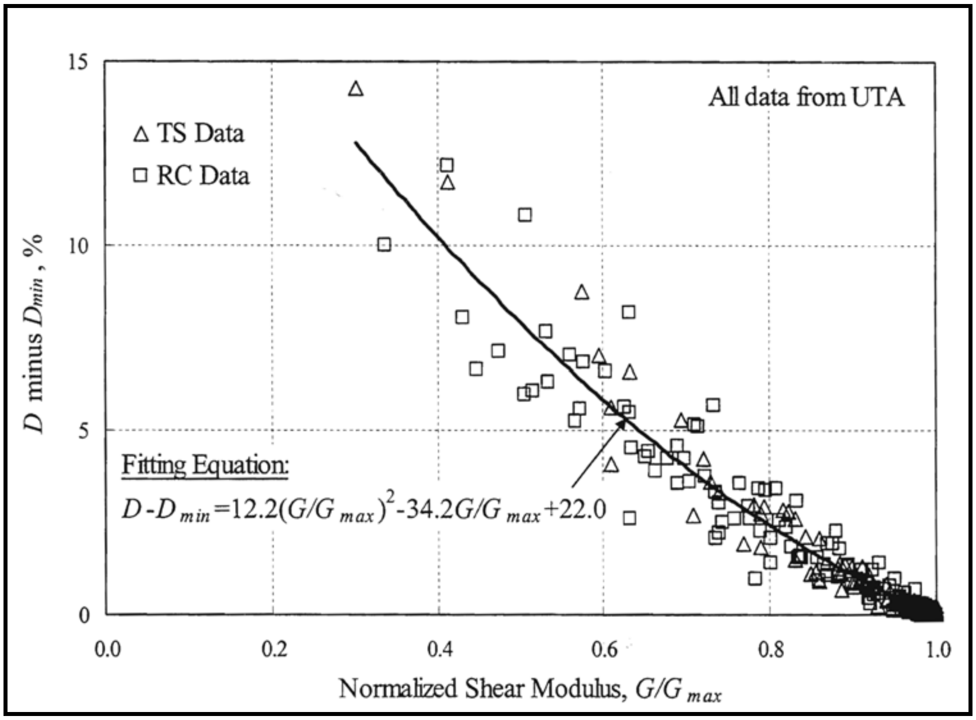


Figure 12-28, (D – D_{min}) vs. (G/G_{max}) Relationship (Andrus et al., 2003)

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	$(K_2)_{max}$ ≈ 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. $$\overline{\boldsymbol{V}}_s^{'} = \overline{\boldsymbol{V}}_s(1-0.20)$$ 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. $$\overline{\boldsymbol{V}}_s^{'} = \overline{\boldsymbol{V}}_s(1-\boldsymbol{COV_{VS}})$$ 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: $$\overline{\boldsymbol{V}}_s = \overline{\boldsymbol{V}}_s^{'}$$ 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 $\bar{s}_u$ < 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 $\bar{\tau} = \bar{s}_u$ < 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 ($\bar{\tau} = \bar{s}_u$) 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.

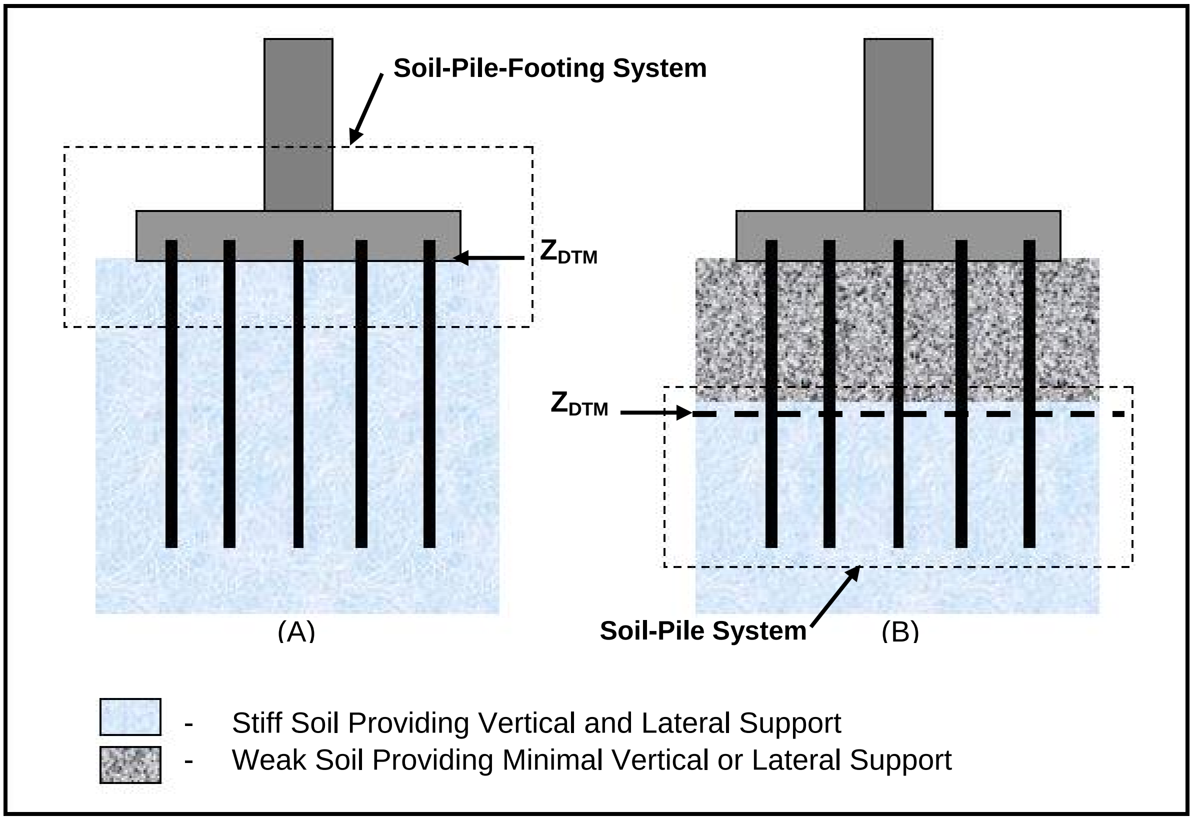


Figure 12-29, Multi-Component Soil-Structure Interaction (MC-SSI)

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 $\bar{V}_S$		Site Class (2, 3)
					(m/s)	(ft/sec)	
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 $\bar{V}_S$ (m/s)	Site Stiffness $\bar{V}_S$ (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 (≈ 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.

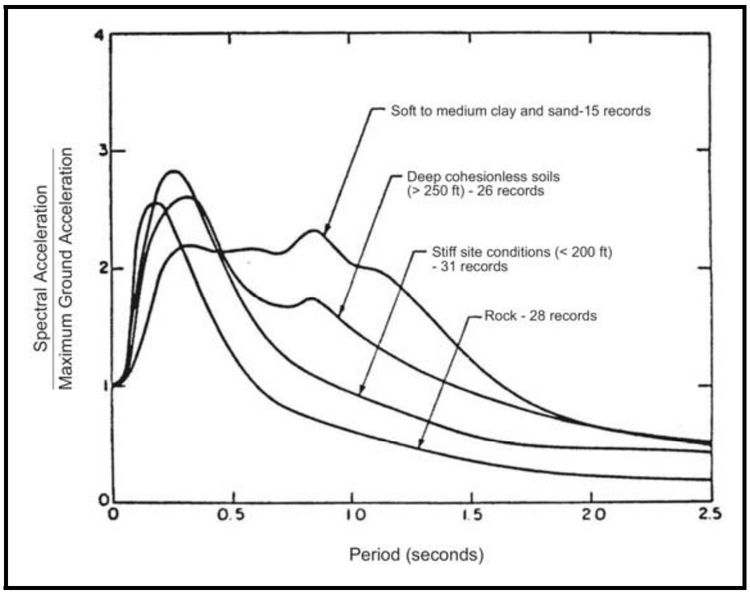


Figure 12-30, Soil Site Effects on Average Normalized Response Spectra (Seed et al., 1976)

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.

$$\boldsymbol{T_N} = \frac{1}{\boldsymbol{f_o}} = \frac{4\boldsymbol{H}}{\boldsymbol{V}^{*}_{s,H}} \quad \text{Equation 12-34}$$

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.

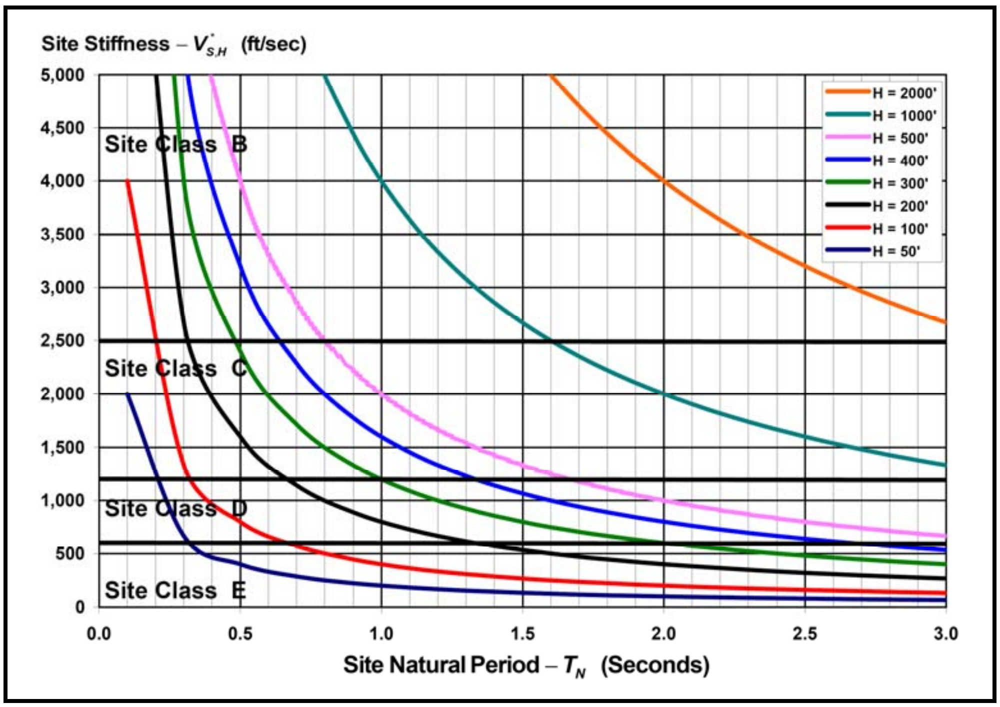


Figure 12-31, Site Natural Period (T_N)

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.

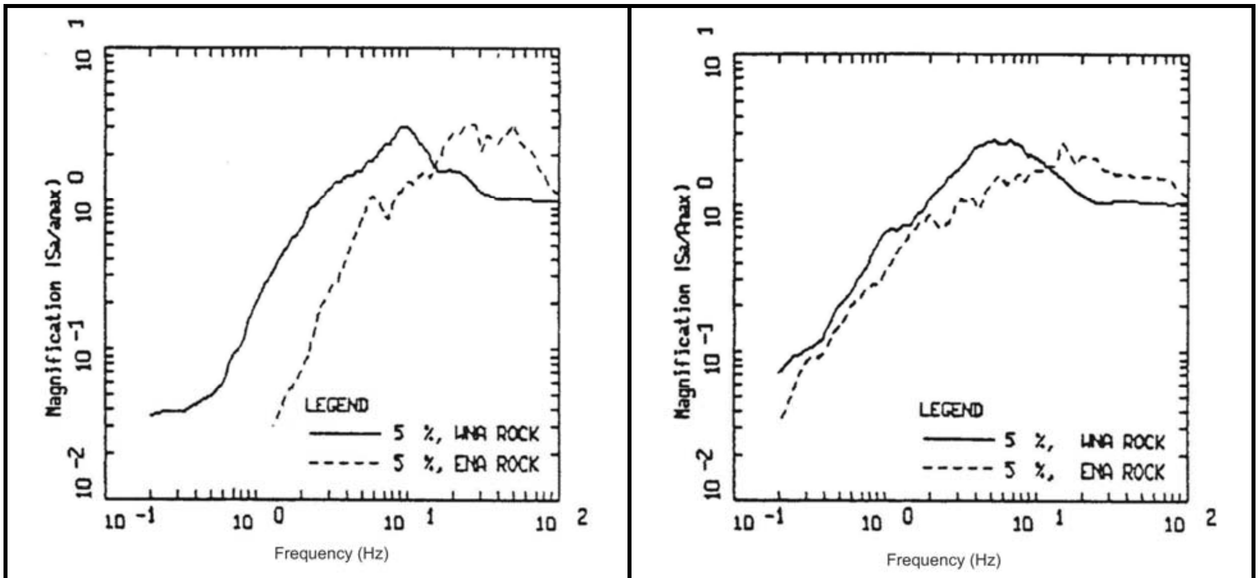
It is equally important to know the fundamental period of the structure (i.e. bridge, earth retaining structure, dam, etc.) being designed since structures with periods similar to the period of the ground motion reaching the structure will tend to exert higher seismic loads (demand) and potentially cause 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 Vertical Ground Motion Response Spectra

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.

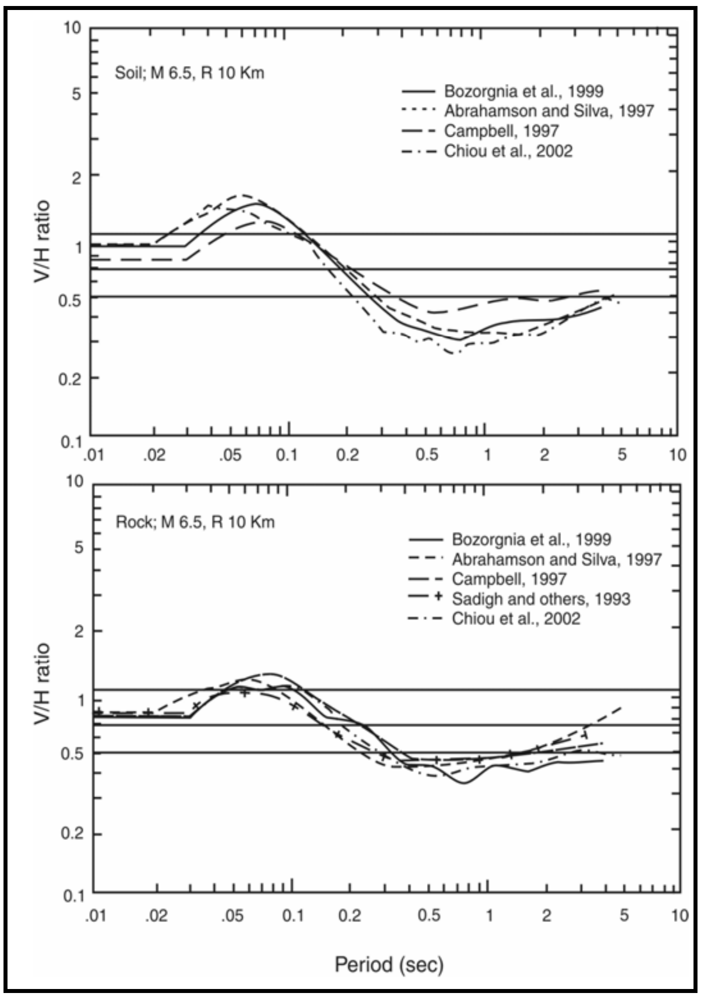


Figure 12-33, Vertical/Horizontal Spectral Ratios vs. Period (Buckle et al, 2005)

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.

$$S_{DS} = F_a S_S \qquad \text{Equation 12-36}$$

$$S_{D1} = F_v S_1 \qquad \text{Equation 12-37}$$

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]				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
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]				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
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.

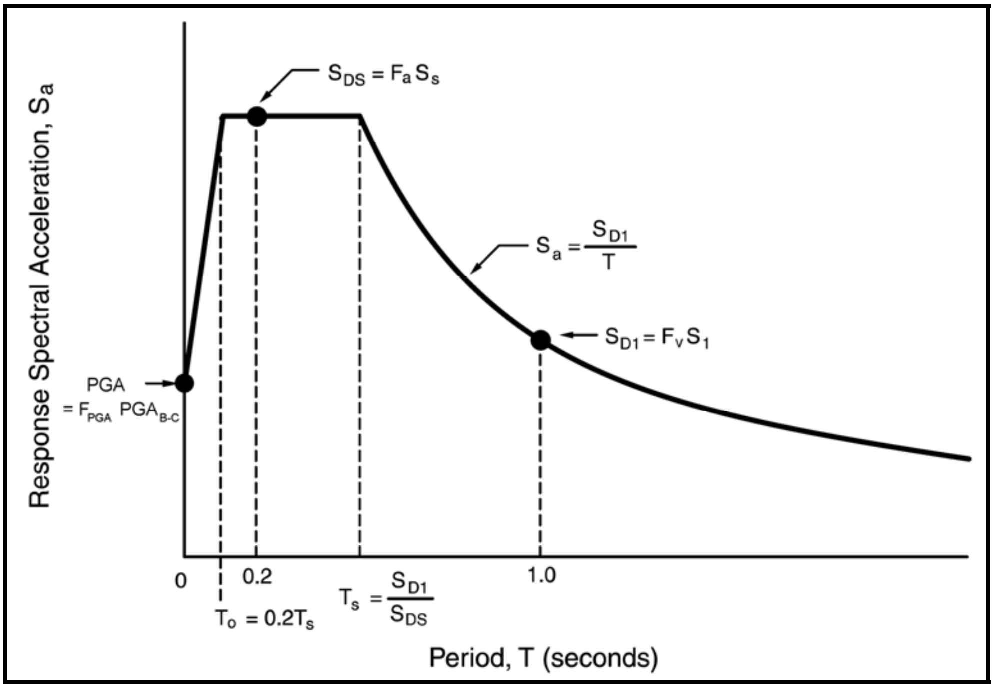


Figure 12-34, Three-Point ADRS Curve

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. $$S_{DS} = F_a S_S \quad \text{Equation 12-36}$$ $$S_{D1} = F_v S_1 \quad \text{Equation 12-37}$$ 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. $$T_o = 0.20T_s \quad \text{Equation 12-38}$$ $$T_s = \frac{S_{D1}}{S_{DS}} \quad \text{Equation 12-39}$$ 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. $$PGA = F_{PGA} \cdot PGA_{B-C} \quad \text{Equation 12-35}$$ 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. $$S_a = PGA + \left[\left(S_{DS} - PGA \right) \left(\frac{T}{T_o} \right) \right] \quad \text{Equation 12-40}$$ 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. $$S_a = \frac{S_{D1}}{T} \quad \text{Equation 12-41}$$ 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.

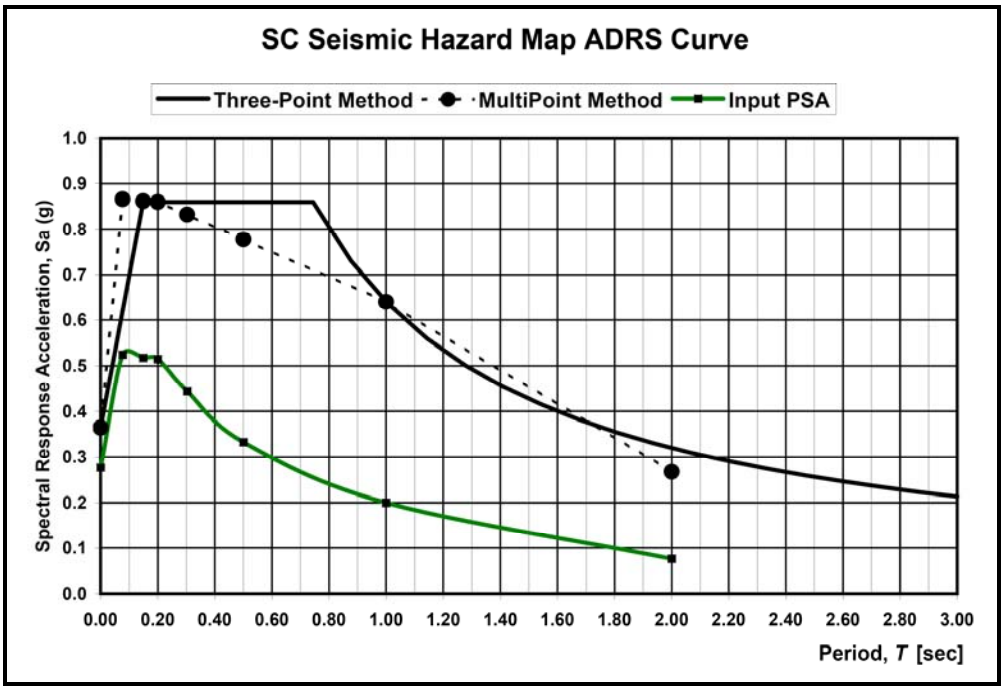


Figure 12-35, Three-Point/Multi-Point ADRS (Site Class=C)

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 ($PGA_{B\text{-}C}$) 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. $$PGA = F_{PGA} \cdot PGA_{B-C} \quad \text{Equation 12-35}$$ Where F_{PGA} and $PGA_{B\text{-}C}$ 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 ≤ 0.20 second is computed using the following equation. $$S_a(T) = F_a S_{\leq 0.20} \quad \text{Equation 12-42}$$ 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 ≤ T ≤ 3.0 second is computed using the following equation. $$S_a = F_v S_{\geq 1.0} \quad \text{Equation 12-43}$$ 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.

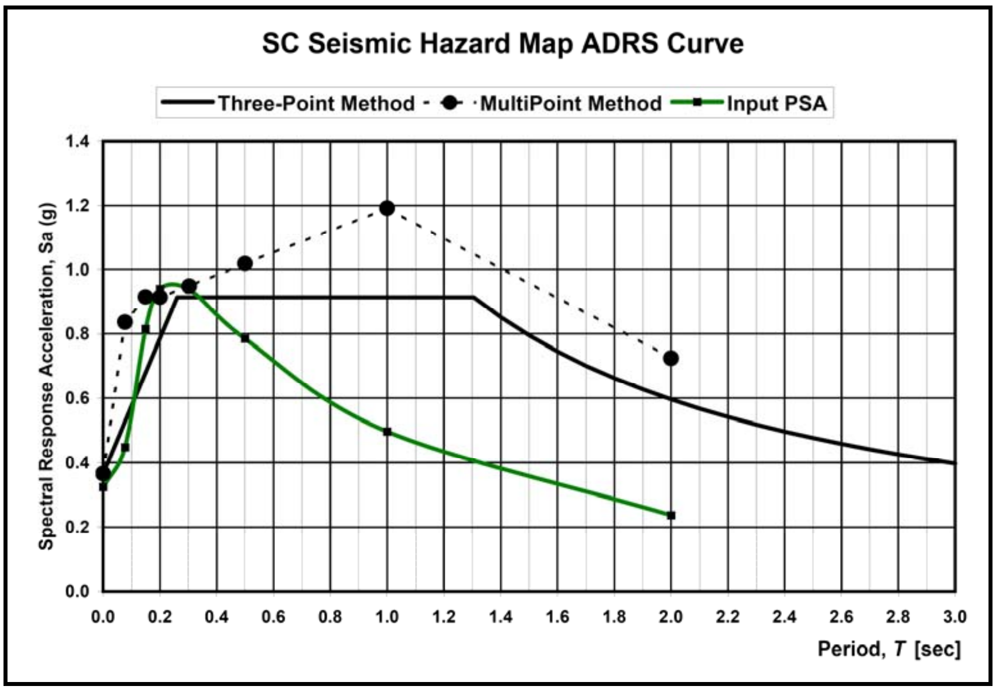


Figure 12-36, Three-Point and Multi-Point Method Comparison (Site Class=E)

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.

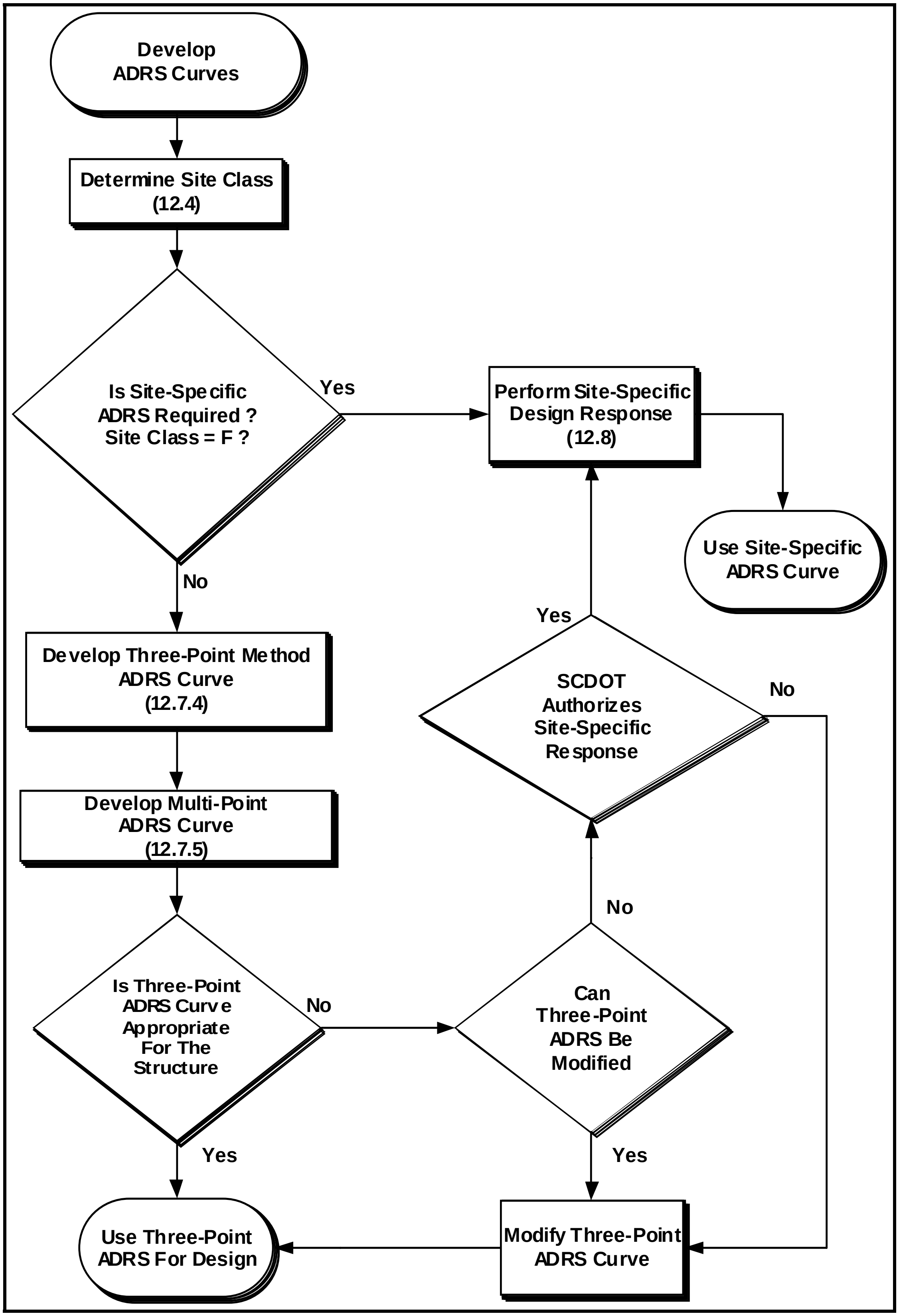


Figure 12-37, ADRS Curve Development Decision Chart

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

<table>
<tr><th>Step</th><th>Procedure Description</th></tr>
<tr><td>1</td><td>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.</td></tr>
<tr><td>2</td><td>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.</td></tr>
<tr><td>3</td><td>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.

$$S_a = \frac{n}{e^T} \qquad \text{Equation 12-44}$$

Where T is the period in seconds and n is a non-dimensional curve fitting number that is adjusted as required.</td></tr>
<tr><td>4</td><td>For periods, T, less than or equal to T_o, the design spectral response acceleration S_a shall be given by the following equation.

$$S_a = PGA + \frac{T(S_{DMax} - PGA)}{T_o} \qquad \text{Equation 12-45}$$

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.</td></tr>
<tr><td>5</td><td>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.</td></tr>
</table>

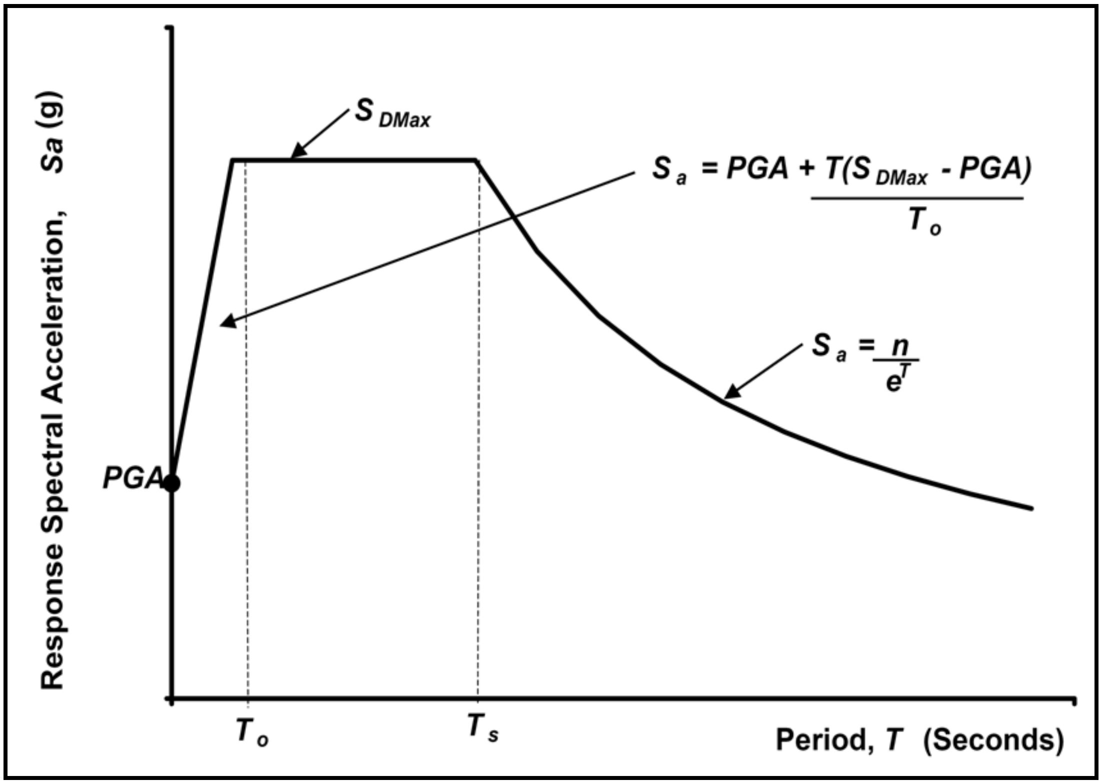


Figure 12-38, Site-Specific Horizontal ADRS Curve Construction

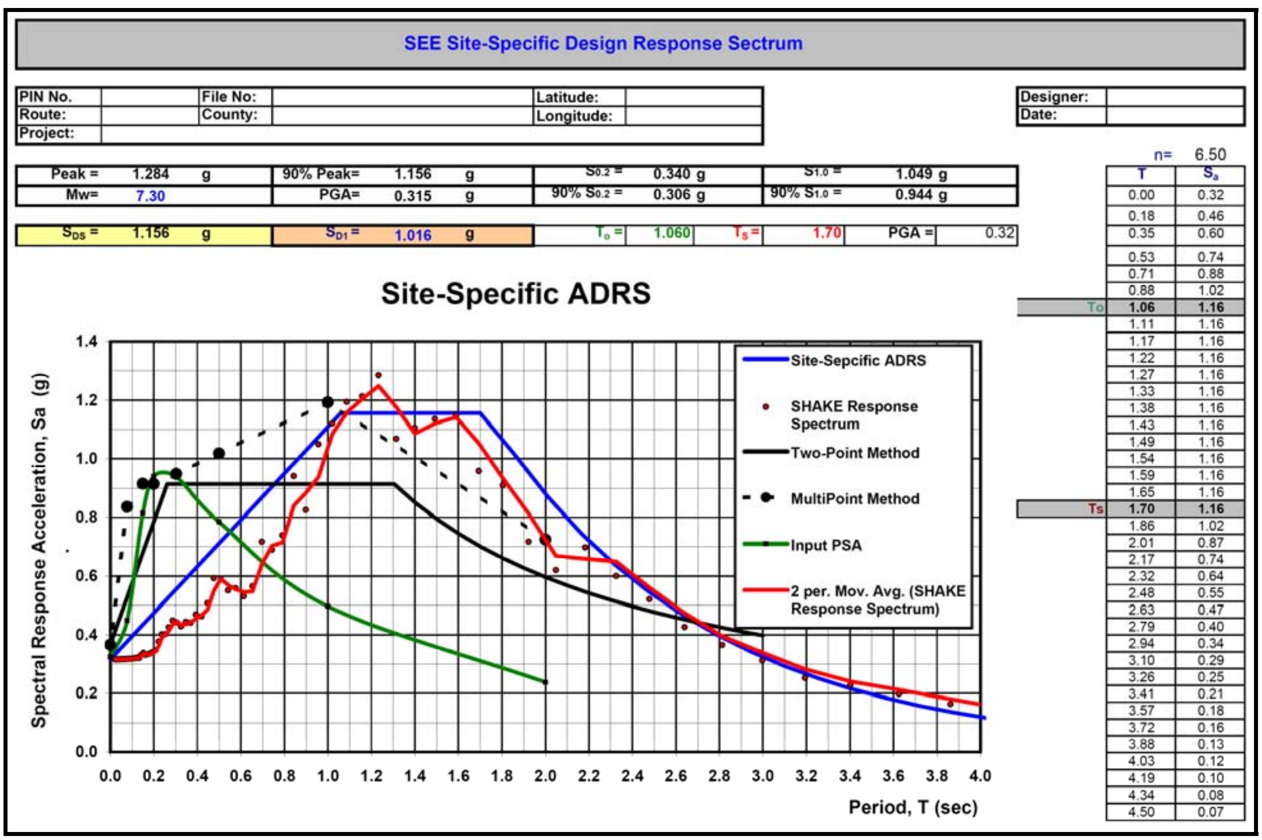


Figure 12-39, Site-Specific Horizontal ADRS Curve (Site Class E)

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 ($D_{a5\text{-}95}$) can be computed by the following equation.

R < 10 km:

Equation 12-46

$$\ln\left(D_{a5-95}\right) = \ln\left[\frac{\left(\dfrac{\exp\left(5.204 + 0.851\left(M_W - 6\right)\right)}{10^{\left(1.5M_W + 16.05\right)}}\right)^{\left(-\frac{1}{3}\right)}}{15.7x10^{6}}\right] + 0.8664$$

R ≥ 10 km:

Equation 12-47

$$\ln\left(D_{a5-95}\right) = \ln\left[\frac{\left(\dfrac{\exp\left(5.204 + 0.851\left(M_W - 6\right)\right)}{10^{\left(1.5M_W + 16.05\right)}}\right)^{\left(-\frac{1}{3}\right)}}{15.7x10^{6}} + 0.063\left(R - 10\right)\right] + 0.8664$$

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 ($D_{a5\text{-}95}$) 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 ($D_{a5\text{-}95}$).

Equation 12-48

$$\ln\left(D_{a5-95}\right) = \ln\left[\frac{\left(\dfrac{\exp\left(2.79 + 0.82\left(M_W - 6\right)\right)}{10^{\left(1.5 M_W + 16.05\right)}}\right)^{\left(-\frac{1}{3}\right)}}{15.68 \cdot 10^{6}} + 0.15R + 2.53 - 0.0041\overline{V}_S + 1.2 \cdot 10^{-3} Z_{HR}\right] + \varepsilon$$

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.

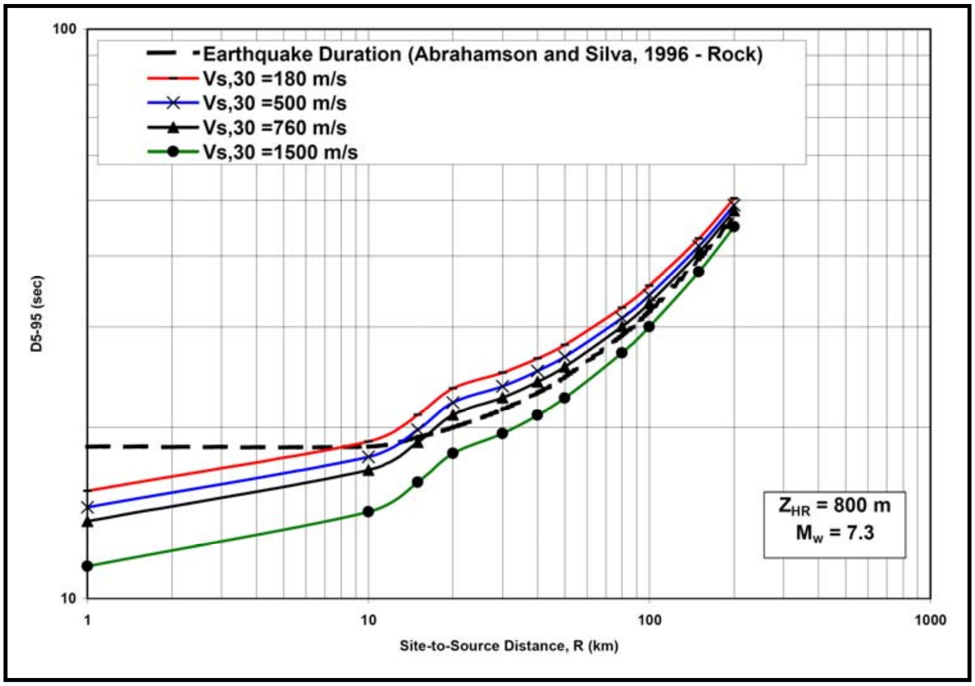


Figure 12-40, Effects of Site Stiffness on Earthquake Duration

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.

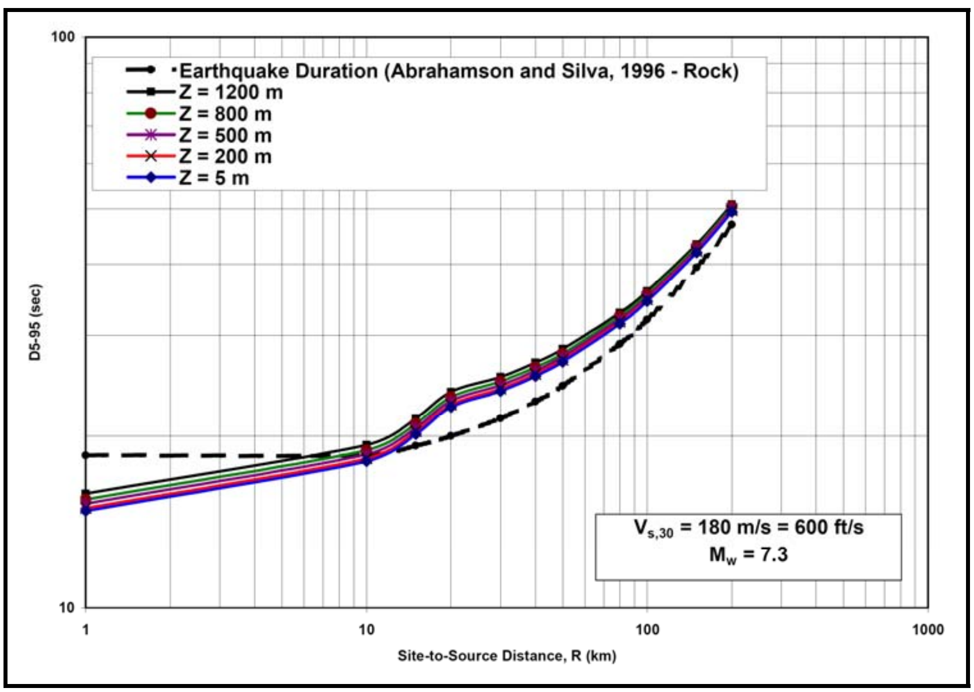


Figure 12-41, Effects of Depth-to-Hard Rock on Earthquake Duration

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.

$$\boldsymbol{V_{Peak}} = \boldsymbol{PGV} = 55\boldsymbol{F_v}\boldsymbol{S}_1 \qquad \text{Equation 12-46}$$

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.

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Chapter 13

GEOTECHNICAL SEISMIC HAZARDS

FINAL

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

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CHAPTER 13

GEOTECHNICAL SEISMIC HAZARDS

13.1 INTRODUCTION

The screening, identification, and evaluation of geotechnical seismic hazards at a project site are an integral part of geotechnical earthquake engineering. The effects of these hazards must be taken into consideration during the design of geotechnical structures such as bridge foundations, retaining walls, and roadway embankments. Geotechnical seismic hazards can generally be divided into those that are associated with losses in soil shear strength and stiffness, seismic ground shaking (i.e. accelerations), and seismic induced lateral ground movements and settlement. Losses in the soil shear strength in South Carolina are primarily due to cyclic liquefaction of loose cohesionless soils and secondarily due to cyclic softening of cohesive soils. Seismic accelerations can create instability due to increased driving forces as a result of increased static active soil pressures. Seismic induced lateral ground movement can occur in sloping ground conditions where the increased driving forces exceed the soil shear strength. Seismic settlement can be either the result of cyclic liquefaction of cohesionless soils, densification/compression of unsaturated soils, or compacted fill materials.

The procedures for analyzing soil Shear Strength Loss (SSL) and associated geotechnical seismic hazards such as flow slide failure, lateral spread, and seismic slope instability are provided in this Chapter. Methods of computing horizontal seismic accelerations based on peak horizontal ground accelerations (PHGA = PGA) and seismic displacements are also provided in this Chapter. Methods of computing seismic active and passive soil pressures on earth retaining structures (ERS) are provided in Chapter 14. Procedures for evaluating seismic settlement due to either cyclic liquefaction or densification/compression of unsaturated soils are presented in this Chapter.

SCDOT recognizes that the methods presented in this Manual may not be the only methods available, particularly since geotechnical earthquake engineering is developing at a very rapid pace as earthquakes around the world contribute to the study and enhancement of analytical methods for geotechnical seismic hazard evaluation. Because geotechnical earthquake engineering in South Carolina (and CEUS) is at the very early stages of development, the overall goal of this Chapter is to establish a state-of-practice that can evolve and be enhanced as methodologies improve and regional (CEUS) experience develops. Methods other than those indicated in this Manual may be brought to the attention of the Pre-construction Support - Geotechnical Design Section (PCS/GDS) for consideration on a specific project or for consideration in future updates of this Manual.

Geotechnical seismic hazards such as fault rupturing and flooding (tsunami, seiche, etc.) are not addressed in this Chapter since current views suggest that the potential for these types of hazards in the Eastern United States is very low. If there is any evidence of faults traversing a project site that have been active within the Holocene epoch (10 thousand years ago to present day) it should be brought to the attention of the PCS/GDS.

South Carolina geology and seismicity, discussed in Chapter 11, will have a major impact on the evaluation of soil SSL and should be well understood when evaluating geotechnical seismic hazards. Earthquake shaking parameters will have a direct effect on the magnitude and extent of the deformations caused by geotechnical seismic hazards. Earthquake shaking parameters such as the moment magnitude (M_W), site-to-source distance (R), duration (D), and peak horizontal ground accelerations (PGHA or PGA) must be determined based on the design earthquake (FEE or SEE) under evaluation as described in Chapter 12. Geotechnical seismic hazards that may affect the design of transportation structures are described in the following Sections and analytical methods are presented to evaluate their potential and magnitude. The effects of geotechnical seismic hazards on the geotechnical design of bridge foundations, abutment walls, earth retaining systems, and other miscellaneous structures are discussed in Chapter 14.

13.2 GEOTECHNICAL SEISMIC HAZARD FAILURE MODES

In order to evaluate the potential for the various geotechnical seismic hazards at a project site, it is important to understand the various modes of failure that have been documented through case histories. Geotechnical seismic hazard modes of failure can be generally categorized as: Global Hazards, Localized Hazards, or Seismic Acceleration Hazards. These geotechnical seismic hazard categories are discussed in the following Sections.

13.2.1 Global Hazards

Global hazards are those failures that result in large-scale site instability in the form of translational/rotational instability and/or flow sliding. These hazards may begin as translational/rotational instability and then trigger a flow slide. Displacements associated with global hazards are the result of soil SSL combined with seismic inertial and/or gravitational driving forces. Displacements associated with translational/rotational instability can be the result of any of the following three instability cases that have also been summarized in Table 13-1.

Case 1. Instability due to soil SSL and static gravitational driving forces (Post-Earthquake condition).

Case 2. Instability due to soil SSL, static gravitational driving forces, and seismic inertial driving forces.

Case 3. Instability due to static gravitational driving forces and seismic inertial driving forces with no loss in soil shear strength

Table 13-1, Global Hazard Instability Cases

Contributors to Instability	Instability Types		
	Case 1	**Case 2**	**Case 3**
Soil SSL	X	X	
Static Gravitational Driving Forces	X	X	X
Seismic Inertial Driving Forces		X	X

A common example of Case 1 instability is a flow slide failure that occurs after the earthquake (post-earthquake). Flow slide failures are the most catastrophic form of ground failures. Sites susceptible to flow failure typically have ground slopes greater than 5 percent grade and are continuous over large areas of soils that are contractive and susceptible to cyclic liquefaction (Section 13.6.1 – Sand-Like Soil). These failures result from post-earthquake instability when the soil shear strength resisting force required for post-earthquake static equilibrium of the soil mass is less than the static gravitational driving force. Flow slide failure is typically confirmed by screening for contractive soils that are susceptible to soil SSL, evaluating triggering of soil SSL, and then evaluating instability by using conventional limit-equilibrium static slope stability methods.

Lateral spread is an example of Case 2 instability that results in lateral ground displacements. Lateral spread typically occurs on gently sloping sites (< 5 degrees) where large blocks of soil displace towards the free-face of a channel or incised river. The displacements result from losses in soil shear strength due to cyclic liquefaction of cohesionless soils combined with static shear stresses and inertial forces induced by the earthquake. Large displacements up to 30 feet have been reported from case studies. Lateral spread can damage bridge foundations and utilities that are located on or across the failure path. Bridge foundations and abutments are particularly susceptible to damage because of their location being near the free-face of a water-way boundary and consequently the large soil pressure loads from the displaced soil mass are applied to the bridge structure and foundation components.

Cases 2 and 3 are typically seismic instability failures that are characterized by translational or rotational slope failure that occurs during earthquake shaking. Translational/rotational instability are typically evaluated by screening for soils susceptible to SSL, evaluating triggering of soil SSL (if applicable), and evaluating instability by using conventional limit-equilibrium pseudo-seismic slope stability methods with appropriate soil shear strengths (accounting for soil SSL) and seismic acceleration coefficients. Deformations are typically evaluated by using Newmark's rigid sliding block displacements methods.

13.2.2 Localized Hazards

Localized hazards include those hazards that occur in isolated areas of failure such as loss of foundation capacity, excessive ground settlement, and lateral displacements.

Foundation capacity failure is the result of SSL of the soils that support either shallow or deep foundations. Shallow foundations are susceptible to bearing capacity failure. Deep foundations are susceptible to vertical foundation movements due to reduced pile capacity to support the structure loads plus any downdrag loads, lateral movements due to loss of lateral support, and pile damage from displacement induced stresses. These hazards are typically evaluated by screening for soils susceptible to soil SSL and using the residual soil shear strengths to compute the resulting margin of safety and estimate the performance of the structure.

Ground settlement is due to volumetric strain (consolidation) (Section 13.18) that results from the earthquake shaking. The settlement can be due to either seismic densification/compression of unsaturated soils or fills and/or seismic consolidation resulting from excess pore water pressure relief of cohesionless soils that have undergone cyclic liquefaction. There may be ground surface manifestations in the form of sand-boils as excess pore water pressure dissipates to the ground surface during cyclic liquefaction. Alternatively, water may get trapped under non-liquefiable capping soil layers above the cyclic liquefiable soils that will affect the rate of soil subsidence and may trigger other hazards due to loss in soil shear strength at these interfaces.

Localized lateral ground displacements can be the result of ground oscillation. Ground oscillation generally occurs on level ground with liquefied soils at a depth that causes a de-coupling of the soils from the ground surface to the top of the liquefied soils. The de-coupling of the surface soils allows permanent displacements at the ground surface that are usually small and disordered in magnitude and direction. Cyclic liquefaction of the subsurface cohesionless soils has been linked to larger amplitude oscillations that in-turn affect the spectral response accelerations used for design by increasing the spectral accelerations for periods greater than one second (Youd and Carter, 2005). `

13.2.3 Seismic Acceleration Hazards

ERSs such as bridge abutments, gravity walls, cantilever walls, etc. (Chapter 18) are the most susceptible to damage resulting from seismic accelerations that induce inertial loads. Seismic inertial loads can cause damage to the structure as described below:

1. Static active earth pressures plus seismic inertial loads can increase lateral earth pressures on ERSs which can result in failure due to deformations that exceed the performance limits or structural capacity of the ERS. Failure may manifest itself in the form of lateral translations, rotations, overturning, or structural failure. Failure of tie-back systems may jeopardize the integrity of the whole structure. Increased bearing loads at the toe of shallow foundations may exceed the bearing capacity of the soil causing rotational displacement or bearing failure.

2. Static passive earth pressure resistance to lateral loads can be reduced due to seismic inertial loads that can result in failure of the ERS by allowing forces from either seismic active soil pressures or inertial forces from the structure to cause large translational displacements.

3. Global limit-equilibrium instability of the structure resulting in rotational or translational deformations that may exceed the ERS performance limits or structural capacity.

13.3 GEOTECHNICAL SEISMIC HAZARD EVALUATION PROCESS

The effects of geotechnical seismic hazards must be considered in the design of all bridges, ERSs, roadway embankments, and other transportation structures where poor performance could endanger the lives and safety of the traveling public. The effectiveness of state highways in South Carolina depends on proper evaluation of the geotechnical seismic hazard and design to meet the performance requirements established in Chapter 10 for bridges, roadway embankments, ERSs and other transportation structures.

The geotechnical seismic hazard evaluation begins with an evaluation of the earthquake shaking parameters that are used to define the intensity and duration of the earthquake at the project site. A summary of the earthquake shaking parameters that will be used during the geotechnical seismic hazard evaluation is presented in Section 13.3.1. The geotechnical seismic hazard evaluation process then proceeds to the screening and identifying of the subsurface soils that have the potential to experience soil SSL. The soil SSL evaluation process is presented in Section 13.3.2. Once the potential for soil SSL has been identified, the potential failure modes of the geotechnical seismic hazards presented in Section 13.2 can be evaluated. For purposes of evaluating the different geotechnical seismic hazard failure modes, the site conditions of *level ground sites* (or gentle sloping, < 5 degrees) and *steeply sloped ground sites* (≥ 5 degrees) are used. Detailed criteria for defining *level ground sites* and *steeply sloped ground sites* are provided in Sections 13.7 and 13.9, respectively.

The effects of the geotechnical seismic hazards on the stability and performance of project sites, embankments and slopes are addressed in this Chapter. The seismic design of bridge foundations, bridge abutments, and ERSs is addressed in Chapter 14.

Provided in Figure 13-1 is a flow chart of the overall geotechnical seismic hazard evaluation process described previously. The processes presented in this Manual are meant to serve as a guide in the evaluation and assessment of geotechnical seismic hazards. It is by no means the only approach that can be used; as a minimum, it should serve as a point of reference to understanding the layout of the following Sections in this Chapter.

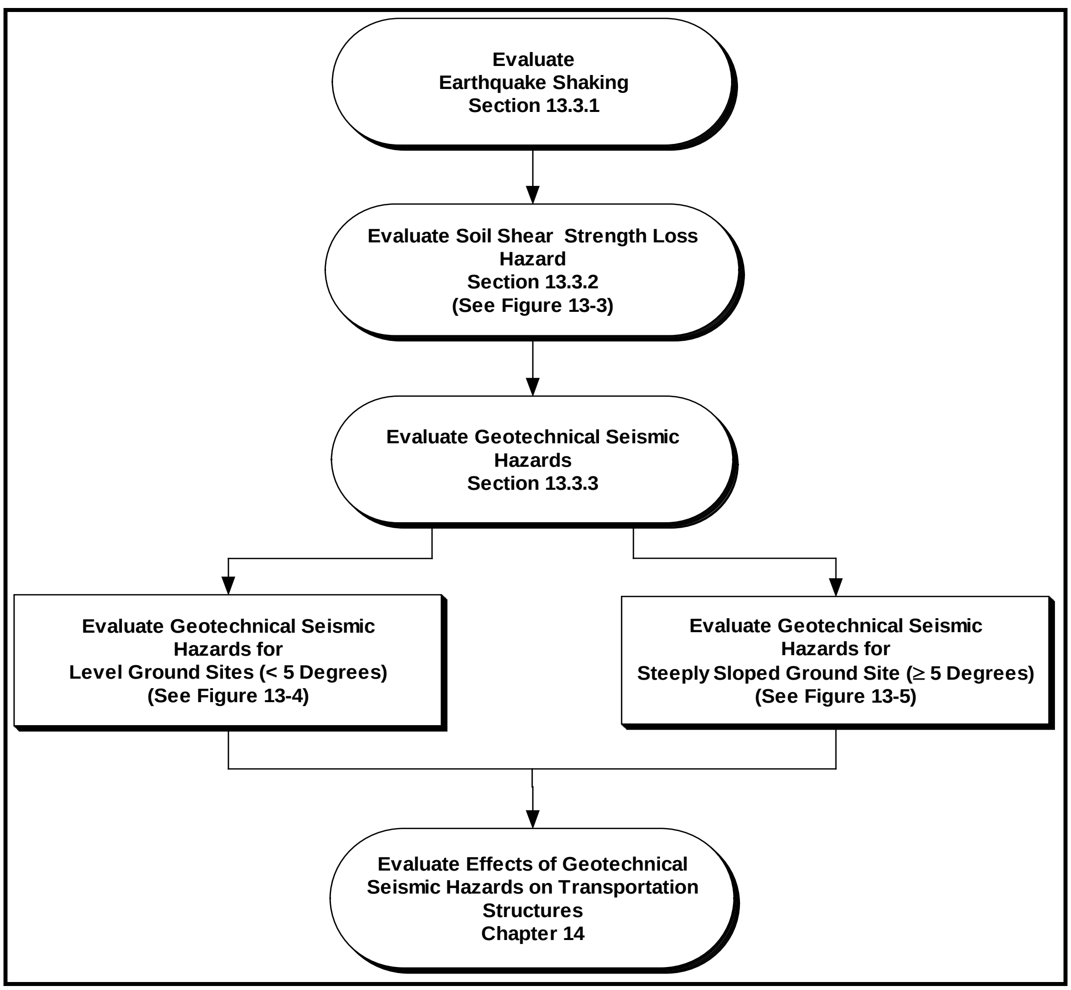


Figure 13-1, Geotechnical Seismic Hazard Evaluation Process

13.3.1 Earthquake Shaking Evaluation Process

Geotechnical seismic hazards are triggered by the intensity and duration of the earthquake shaking at the project site. The intensity and duration of the earthquake shaking is primarily dependent on the size and location of the earthquake and the characteristics of the site. Chapter 12 - Geotechnical Earthquake Engineering provides the methodology for the assessment of the earthquake shaking at a project site. The earthquake shaking can be quantitatively assessed by the moment magnitude of the earthquake (M_W), site-to-source distance (R), peak horizontal ground acceleration at the ground surface (PHGA = PGA), the duration of the earthquake (D), and design spectral acceleration at 1 second (S_{D1}). The level of shaking at the project site is directly proportional to the potential for geotechnical seismic hazard damage. Project sites that are closer to the earthquake source experience higher levels of shaking; therefore, more damage can occur from geotechnical seismic hazards when compared to project sites further away.

13.3.2 Soil Shear Strength Loss Hazard Evaluation Process

Soil SSL that is induced by the earthquake shaking can produce severe damage as a result of the various geotechnical seismic hazard failure mechanisms as described in Section 13.2. The soil SSL hazard evaluation process has three components: (1) Evaluating soil SSL susceptibility at the project site; (2) Evaluating soil SSL triggering potential of the earthquake shaking; and (3) Evaluating the effects of soil SSL on the design parameters used to evaluate the geotechnical seismic hazard.

The soil SSL evaluation process begins with the screening for soils that are susceptible to soil SSL for the design earthquake (FEE or SEE) under evaluation. The screening criteria (Section 13.6) consist of three soil categories that are susceptible to soil SSL: Sand-Like soils, Normally Sensitive (NS) Clay-Like soils, and Highly Sensitive (HS) Clay-Like soils. The screening criteria uses standard laboratory soil shear strength testing, in-situ testing, index testing, and site conditions (i.e. water table) to determine if soils are susceptible to SSL. If soils are found not to be susceptible to soil SSL during the screening process, then no further analysis is required of the triggering of soil SSL and an evaluation of geotechnical seismic hazard evaluation can proceed.

The main contributor to catastrophic damage and poor performance of structures has in past case histories been attributed to cyclic liquefaction-induced seismic geotechnical hazards shown in Figure 13-2. Soil SSL due to cyclic liquefaction of Sand-Like soils (Section 13.6.1) has the potential to cause the most damage in the Coastal Plain of South Carolina as evident from the historical cyclic liquefaction case histories presented in Section 13.5.3.

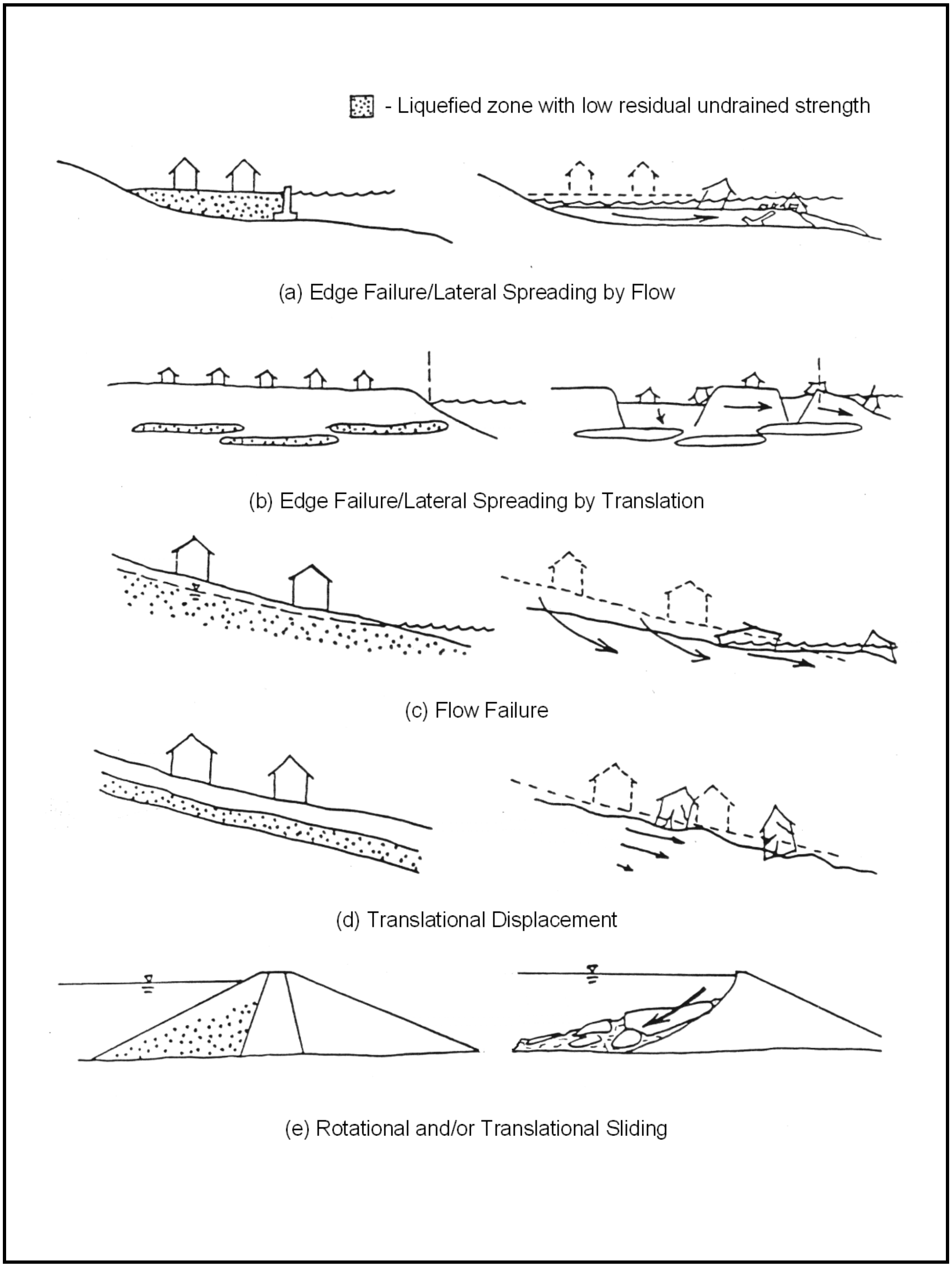


Figure 13-2, Cyclic Liquefaction-Induced Seismic Geotechnical Hazards (Seed et al., 2003)

Soils that are identified as being susceptible to losses in soil shear strength need to be evaluated to determine if the earthquake shaking can trigger (or initiate) the soil SSL. Soil SSL triggering for Sand-Like soils and Clay-Like soils is dependent on the site conditions of level ground or steeply sloped ground. The soil SSL triggering of Clay-Like soils is applicable to both NS Clay-Like soils and HS Clay-Like soils. The overall method for analyzing the triggering of soil SSL for Sand-Like soils and Clay-Like soils consists of determining if the cyclic stresses induced by design earthquake (FEE or SEE) and any initial static shear stresses in the soil (Demand, D) are greater than the soil's cyclic resistance (Capacity, C) based on a specified margin of safety (on-set of soil SSL resistance factor, φ_{SL}). If the soil SSL resistance ratio, $(D/C)_{SL}$, is greater than φ_{SL}, the soil under evaluation has the potential for soil SSL and a reduced shear strength should be used in the evaluation of geotechnical seismic hazards.

The triggering of soil SSL at *Level Ground Sites* (or gently sloping, < 5 degrees) is based on an evaluation of the cyclic stress ratio (CSR = Demand) and the cyclic resistance ratio (CRR = Capacity) of the soils as described in Section 13.7. These sites are assumed to have no initial static shear stresses (τ_{Static}) that would reduce the soil's cyclic resistance capacity (C).

The triggering of soil SSL at *Steeply Sloped Ground Sites* ($\geq$ 5 degrees) is more complex because the initial static shear stress (τ_{Static}) reduces the soil's capacity (C) to resist the soil SSL. The triggering of soil SSL at steeply sloped ground sites consists of first screening for flow failure as described in Section 13.8. If the screening indicates that the soil is contractive and flow failure resistance ratio, $(D/C)_{Flow}$ is greater than φ_{Flow}, then a soil SSL triggering analysis is required as described in Section 13.9. If screening indicates that the site does not have the potential for flow failure within a margin of safety (on-set of flow failure resistance factor, φ_{Flow}) then a soil SSL triggering analysis is not required and soil SSL is less likely to happen. The triggering of soil SSL in Clay-Like soils (NS and HS) can also occur in steeply sloped ground sites due to an increase in static shear stresses that occurs when Sand-Like soils experience cyclic liquefaction. Soil SSL in NS Clay-Like soils causes the soils to have cyclic softening residual shear strength (τ_{rs}) and in HS Clay-Like soils causes the soils to have remolded soil shear strength ($\tau_{remolded}$).

The selection of soil shear strength properties for soils with and without the potential for soil SSL is performed during the geotechnical seismic hazard evaluation. The overall process for evaluating soil SSL is shown in Figure 13-3.

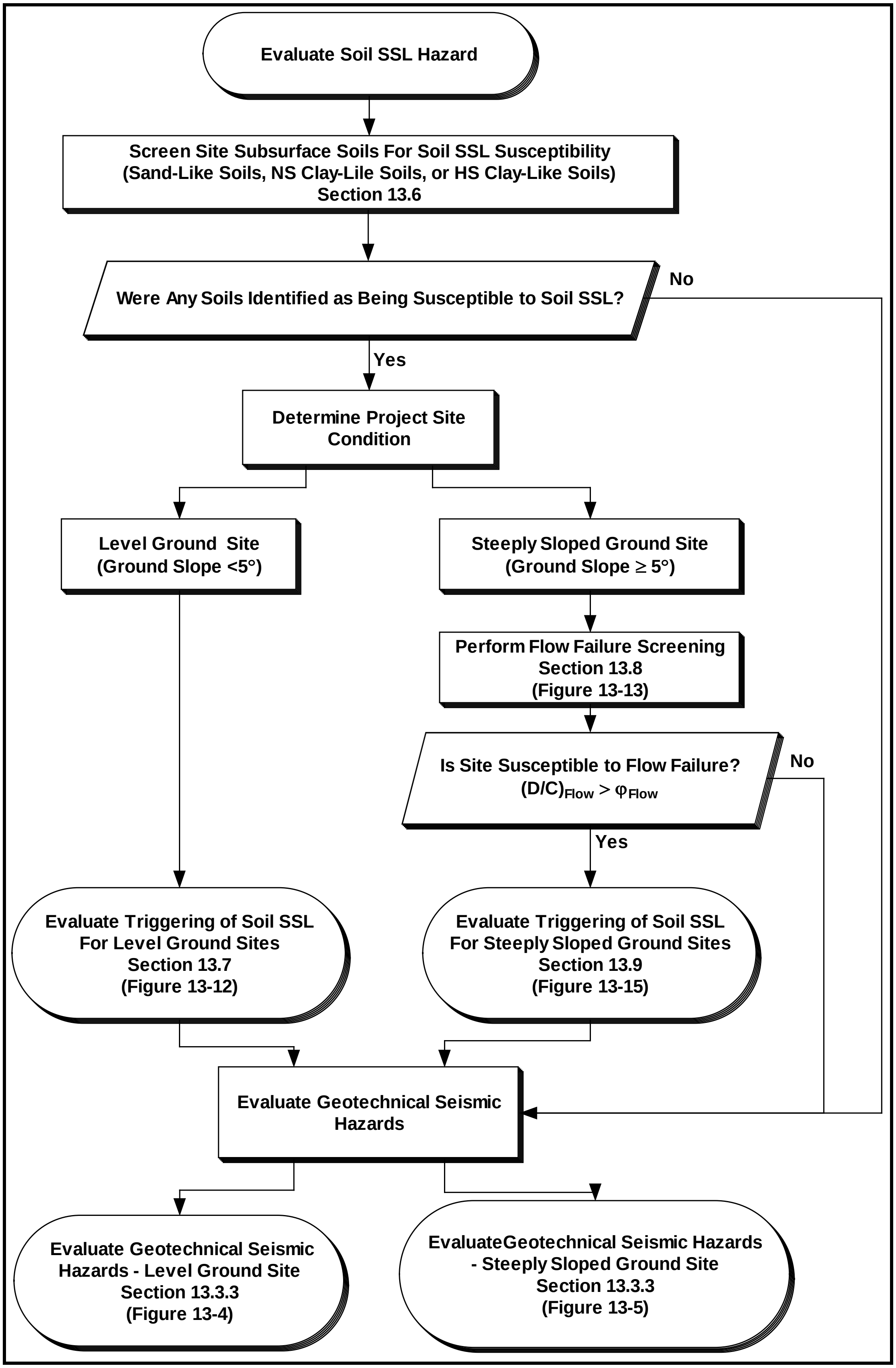


Figure 13-3, Soil SSL Hazard Evaluation Process

13.3.3 Geotechnical Seismic Hazards Evaluation Process

Geotechnical seismic hazards are evaluated after the triggering of soil SSL has occurred. The determination of whether the triggering of soil SSL occurs during the earthquake shaking or after the earthquake shaking is very complex and beyond the scope of the methodology that will be used in the design of standard bridges and typical roadway structures. Therefore, the effects of cyclic liquefaction and cyclic softening shall be assumed to occur during the earthquake shaking and post-earthquake for the evaluation of SSL-induced geotechnical seismic hazards. Cyclic liquefaction of Sand-Like soils shall be assumed to occur instantaneously throughout the full thickness of the Sand-Like soil layer. These fundamental assumptions must be used when selecting soil shear strengths in accordance with Section 13.12.

The overall geotechnical seismic hazard evaluation process for level project sites and steeply sloped project sites is presented in Figures 13-4 and 13-5, respectively.

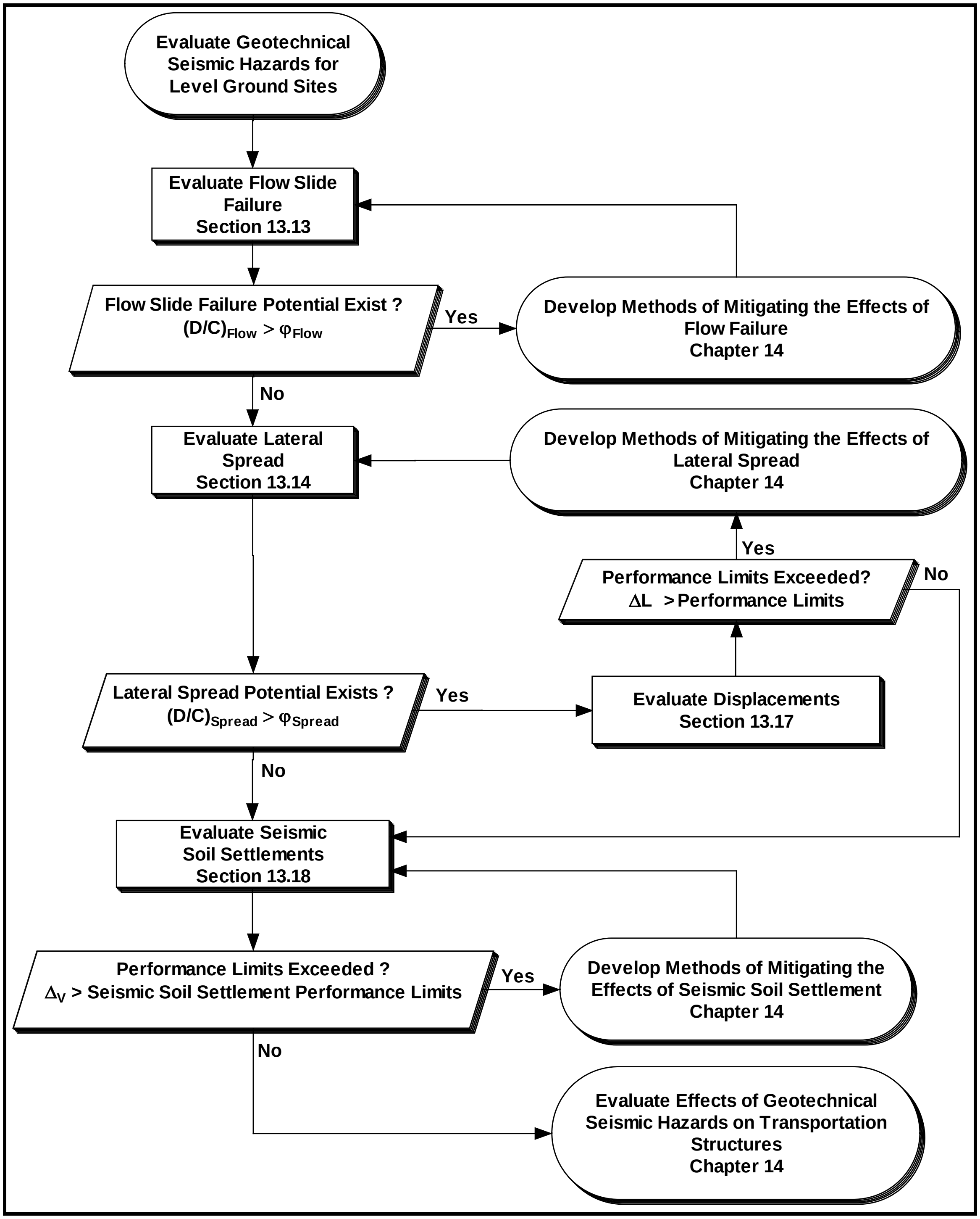


Figure 13-4, Geotechnical Seismic Hazards - Level Ground Sites

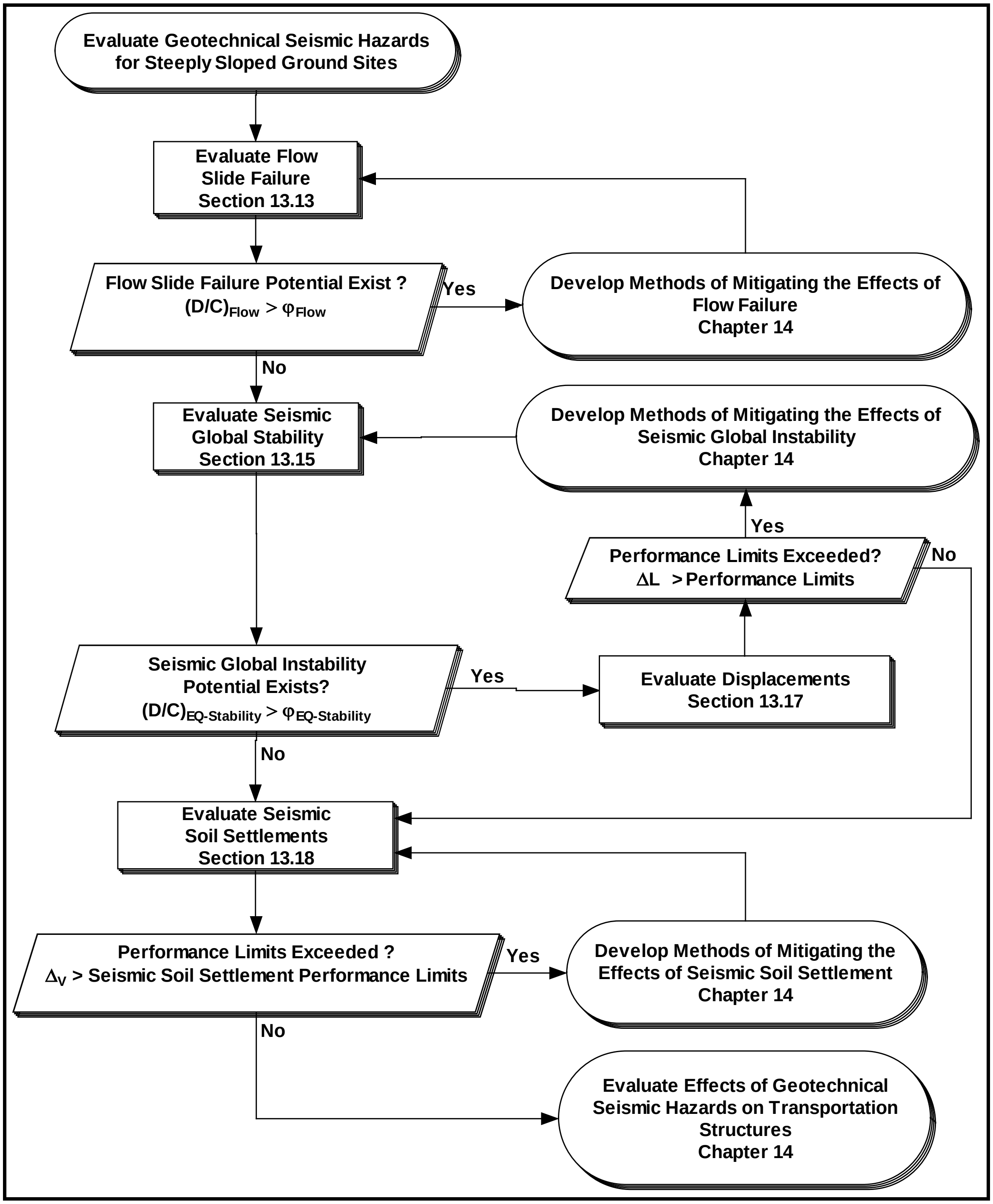


Figure 13-5, Geotechnical Seismic Hazard – Steeply Sloped Ground Sites

13.4 GEOTECHNICAL SEISMIC HAZARD ANALYTICAL METHODOLOGIES

The methodologies presented in this Chapter for evaluating and assessing the impact of the geotechnical seismic hazards on transportation structures are based primarily on limit-equilibrium methods of analyses and empirical/semi-empirical analytical methods that are easily performed and are currently within the state-of-practice of geotechnical earthquake engineering. References have been provided for the design methodologies used in this Manual to allow the designer to become thoroughly familiar with the methodology, its applicability, and its limitations. Within the scope of this Manual, it is not possible to provide sufficient detail and caveats to preclude any misuse of the methods. When necessary, several methods of analyzing the geotechnical seismic hazard have been provided in order to allow for variance in analytical methodologies and to identify trends in results or performance. Several of the methods presented are empirical/semi-empirical based and their applicability to the project site is dependent on the limits of the database used to develop the analytical basis of the method. This Chapter will not address numerical analyses (e.g., finite element, finite difference, etc.), because these methods are typically not performed in the design of standard bridges or typical transportation structures. If numerical analyses are required for a project, the PCS/GDS will provide guidance and/or analytical requirements.

13.5 SOIL SHEAR STRENGTH LOSS MECHANISMS

The mechanism of soil SSL is very complex and has been the subject of much confusion in literature. This is particularly due to the lack of standardization of terminology and the fact that research efforts are still ongoing. Additional confusion has occurred when the method of SSL triggering (static stresses, cyclic loads, etc.) has been used as a means of categorizing the soil SSL mechanism. Current understanding of soil SSL failure mechanisms is based on the study of case histories and laboratory experimentation. One of the problems in the evaluation of field case histories is that more than one geotechnical seismic hazard is typically responsible for the observed failures. This problem can occur when lateral spread movements trigger flow failures and the resulting final deformations observed reflect the influence of all geotechnical seismic hazard failure modes (lateral spread, flow failure, and seismic settlement). Laboratory testing has provided much insight into the mechanisms that trigger soil SSL under a controlled laboratory environment. Laboratory experimentation has limitations in that sample disturbance of the soil structure (i.e. cementation, layering, etc.) can significantly affect the initial and residual soil shear strength results. Another limitation is that laboratory testing can be very complex and is typically not within the standard-of-practice for design of most typical bridge structures. A more detailed explanation of the mechanisms of soil SSL based on field and laboratory observations can be obtained from Robertson and Wride (1997), Kramer and Elgamal (2001), and Idriss and Boulanger (2008). Although the term liquefaction has been used widely in literature (Kramer, 1996 and Robertson and Wride, 1997) to describe several mechanisms of soil SSL, the term liquefaction in this Manual will only be applicable when discussing soil SSL of cohesionless soils that result from cyclic liquefaction.

The predominant soil SSL behavior is used in this Manual to evaluate the soil's susceptibility (Section 13.6) and to determine the most appropriate soil SSL trigger evaluation method for use in geotechnical seismic design (Sections 13.7 for level ground sites and 13.9 for steeply sloped ground sites). Field case histories and laboratory testing have demonstrated that the predominant soil SSL behavior of the majority of the soils can be grouped into either Sand-Like soils that are subject to cyclic liquefaction failure mechanism or Clay-Like soils that are subject cyclic softening failure mechanism. A description of these soil SSL failure mechanisms is provided in the following Sections.

13.5.1 Cyclic Liquefaction of Sand-Like Soils

Cyclic liquefaction of Sand-Like soils is typically responsible for the most damaging geotechnical seismic hazards that affect transportation infrastructure. Potential damage to transportation facilities due to cyclic liquefaction includes loss of bearing capacity, lateral spread, flow failure, excessive settlements, embankment/slope instability, and reduced lateral and vertical carrying capacity of deep foundations. Even though liquefaction can be triggered by non-seismic loadings such as low amplitude vibrations produced by train traffic/construction equipment or by static loads, such as those that might be caused by rapid drawdown, this Manual will focus on liquefaction triggered by seismic earthquake shaking. Cyclic liquefaction typically occurs in Sand-Like soils that are nonplastic, saturated, and have been deposited during the Quaternary Period (past 1.6 million years ago) in a loose state and are subject to strain softening. Typically, the more recent soil deposits have the greatest susceptibility for cyclic liquefaction. Cyclic liquefaction typically begins during an earthquake when the in-situ soil pore water pressure (u_o) increases ($+\Delta u$). As the increased pore water pressure ($u = u_o + \Delta u$) approaches the total overburden stress (σ_{vo}) the effective overburden stress ($\sigma'_{vo} = \sigma_{vo} - u$) will decrease causing a reduction in grain-to-grain contact that brings about a significant decrease in soil shear strength. The reduction in grain-to-grain contact causes a redistribution of soil particles resulting in densification.

Significant lateral soil deformation may occur as a result of reduced soil shear strength of the liquefied soil zone combined with the seismic inertial forces and/or initial static driving forces that are typically the driving force for soil instability. Other surface manifestations of cyclic liquefaction are often associated with the upward flowing of pore water that generates sand boils at the ground surface. Evidence of sand boils occurring at the ground surface have been found throughout the South Carolina Coastal Plains as indicated in Section 13.5.3. The absence of sand boils is not an indication that cyclic liquefaction has not occurred. Sand boils will not occur if cyclic liquefaction occurs and the drainage paths are restricted due to overlying less permeable layers, i.e., the sand immediately beneath a less permeable soil which can loosen due to pore water redistribution, resulting in subsequent flow failure at this interface. Seismic settlement at the ground surface may occur from cyclic liquefaction induced volumetric strain that develops as seismically induced pore water pressures dissipate.

The determination of the onset of cyclic liquefaction either during or post-earthquake shaking is a very complex analytical problem and beyond the scope of typical SCDOT projects. Several case histories have documented that liquefaction can occur during shaking or post-earthquake (Seed, 1986; Kramer and Elgamal, 2001). The onset and manifestation of cyclic liquefaction is primarily dependent on the magnitude, duration, and proximity of the seismic event, the depth of the liquefied soil zone, stratification and relative permeability of the soil layers above and below the liquefied soil zone, and the susceptibility of the soils to liquefy. Consequently, liquefaction will conservatively be assumed to occur during the earthquake shaking.

13.5.2 Cyclic Softening of Clay-Like Soils

Cyclic softening refers to the soil SSL and deformations in Clay-Like soils. Clay-Like soils are typically moist and plastic clays. Cyclic softening occurs when the earthquake-induced cyclic shear stresses exceed the soil's cyclic shear resistance, causing an accumulation of deformations that result in soil SSL in cohesive soils that exhibit strain softening. Cyclic softening of Clay-Like soils typically results in soil SSL that is dependent on the soil's sensitivity (Chapter 7). Soil deformations may occur as a result of reduced soil shear strength of Clay-Like soils combined with the inertial forces and/or initial static driving forces that are typically the driving force for soil instability. The limited case histories in South Carolina have not documented cyclic softening of Clay-Like soils. Cyclic softening of Clay-Like soils is difficult to document because it does not manifest itself as sand boils at the ground surface as has been documented for cyclic liquefaction of Sand-Like soils.

13.5.3 SC Historical Cyclic Liquefaction

There is significant evidence that soil cyclic liquefaction has historically occurred in the CEUS. Soil liquefaction has been found to have occurred as a result of earthquakes in New Madrid, Missouri 1811 – 1812 and in Charleston, South Carolina 1886. The 1886 Charleston earthquake caused the manifestation of large sand boils as a result of cyclic liquefaction. Sand boils were created as the soil pore water, carrying soil particles, was expelled from the soil causing collapse that resulted in a crater at the ground surface. Figure 13-6 shows a sand boil crater that appeared during the 1886 Charleston earthquake.

Figure 13-6, Sand Boil Crater - 1886 Charleston, SC Earthquake (McGee, et al., 1986)

Hayati and Andrus (2008) developed a liquefaction potential map of Charleston, South Carolina based on the 1886 earthquake. The geologic map of the Charleston Peninsula and Drum Island originally developed by Weems et al. (1997) was used by Hayati and Andrus (2008) to indicate locations of liquefaction and ground deformations as shown in Figure 13-7. For a description of the near surface geologic units and a description of the Cases (indicated on the map as 1 – 27) of cyclic liquefaction evidence and permanent ground deformation see Hayati and Andrus (2008).

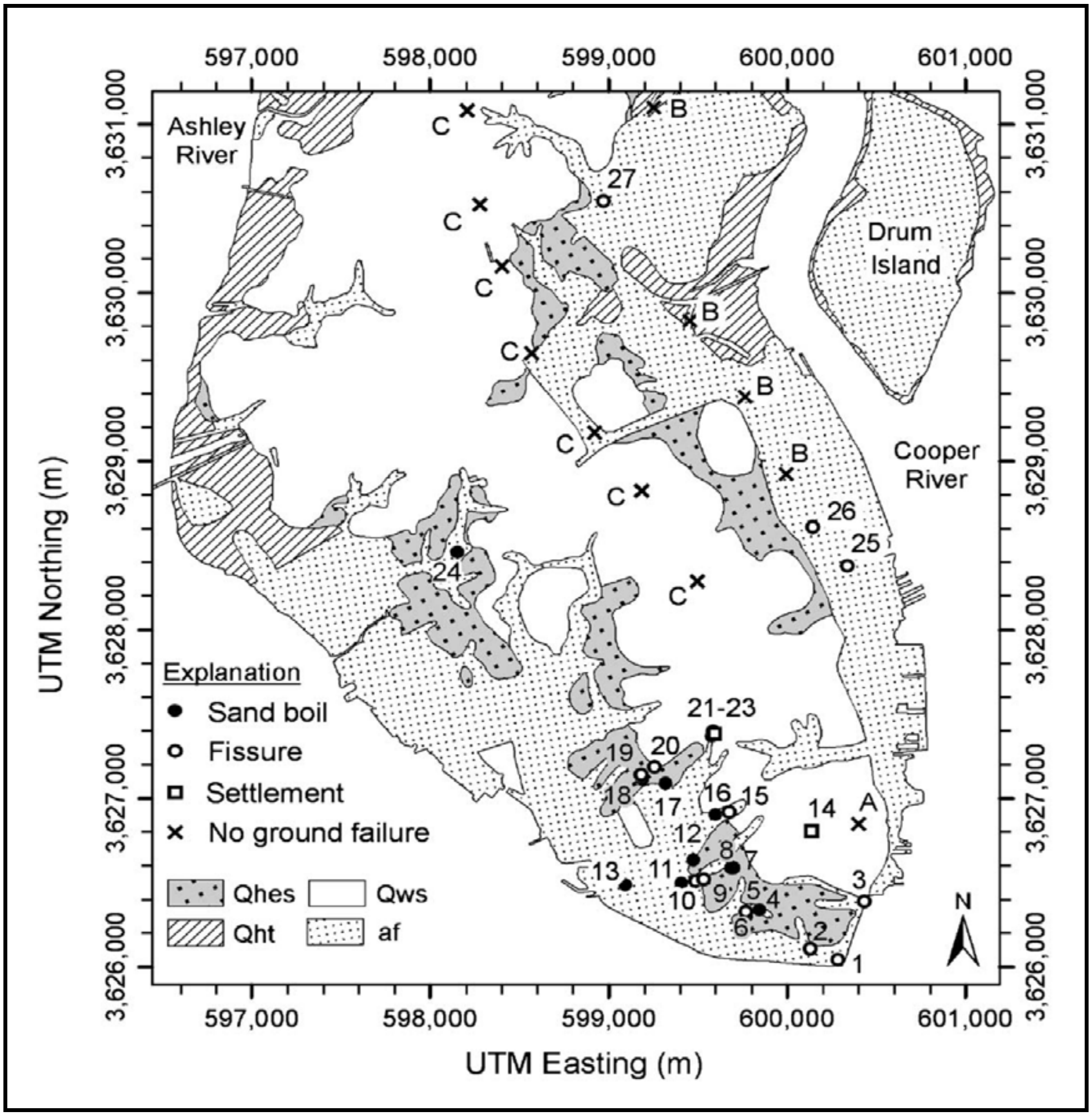
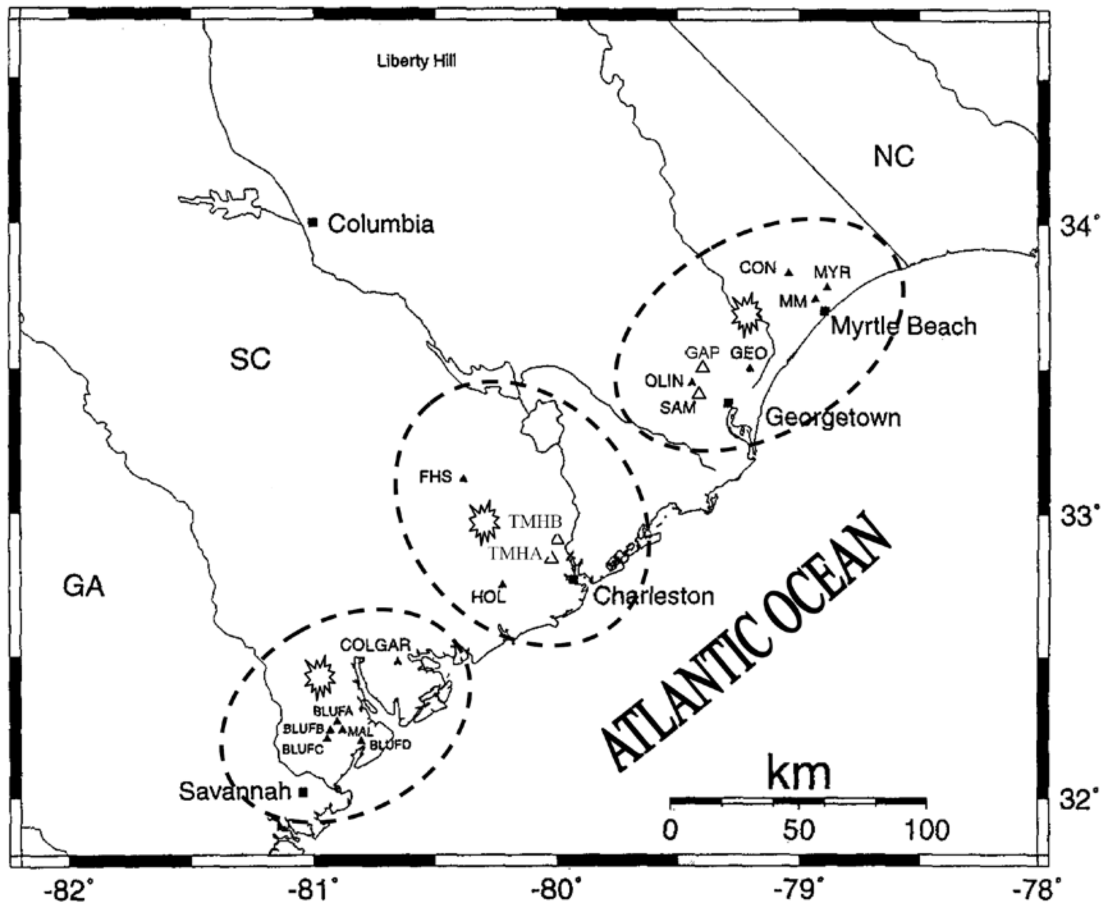


Figure 13-7, 1886 Liquefaction and Ground Deformations Sites (Weems et al., 1997, Hayati and Andrus, 2008 with permission from ASCE)

Paleoliquefaction studies in South Carolina conducted since the mid-1980s have indicated that at least seven episodes of paleoliquefaction have occurred in the past 6,000 years. The earthquakes in the Charleston, SC area appear to have magnitudes greater than 7 and the earthquake cycle suggest a recurrence time of 500-600 years (Talwani and Schaffer, 2001). Paleoliquefaction study site locations in the South Carolina Coastal Plain are shown in Figure 13-8.



Legend:

- Dashed lines enclose three zones of paleoliquefaction along the South Carolina Coastal Plain. The explosion symbols represent three possible inferred epicentral locations.
- Triangles show the locations of sandblows in South Carolina Coastal Plain. Reports of liquefaction features extend to Columbia and Georgetown and to Sand Hills near Liberty Hill.
- Open triangles indicate the locations of in-situ engineering tests for this study.
- Abbreviations are as follows: Bluffton, BLUF; Colony Gardens, COLGAR; Conway, CON; Four Hole Swamp, FHS; Gapway, GAP; Georgetown, GEO; Hollywood, HOL; Malpherous, MAL; Martin Marietta, MM; Myrtle Beach, MYR; Sampit, SAM; and Ten Mile Hill, TMH

Figure 13-8, Coastal Plain Paleoliquefaction Study Sites (adapted from Talwani and Schaffer, 2001).

The USGS maintains a database of published reports of Quaternary faults, liquefaction features, and tectonic features in the CEUS. The USGS database for South Carolina contains the following three sites with liquefaction features: 2657, Charleston, SC; 2658, Bluffton, SC; and 2659, Georgetown, SC. Liquefaction feature 2657 has geologic evidence of the 1886 Charleston earthquake. Liquefaction features 2658 and 2659 have geologic evidence of prehistoric liquefaction that occurred during the late Quaternary Period (Holocene, <10,000 years ago). Figure 13-9 shows a map, prepared by the USGS, of the liquefaction features in South Carolina. The shaded area on the map indicates areas of potential Quaternary and historic liquefaction.

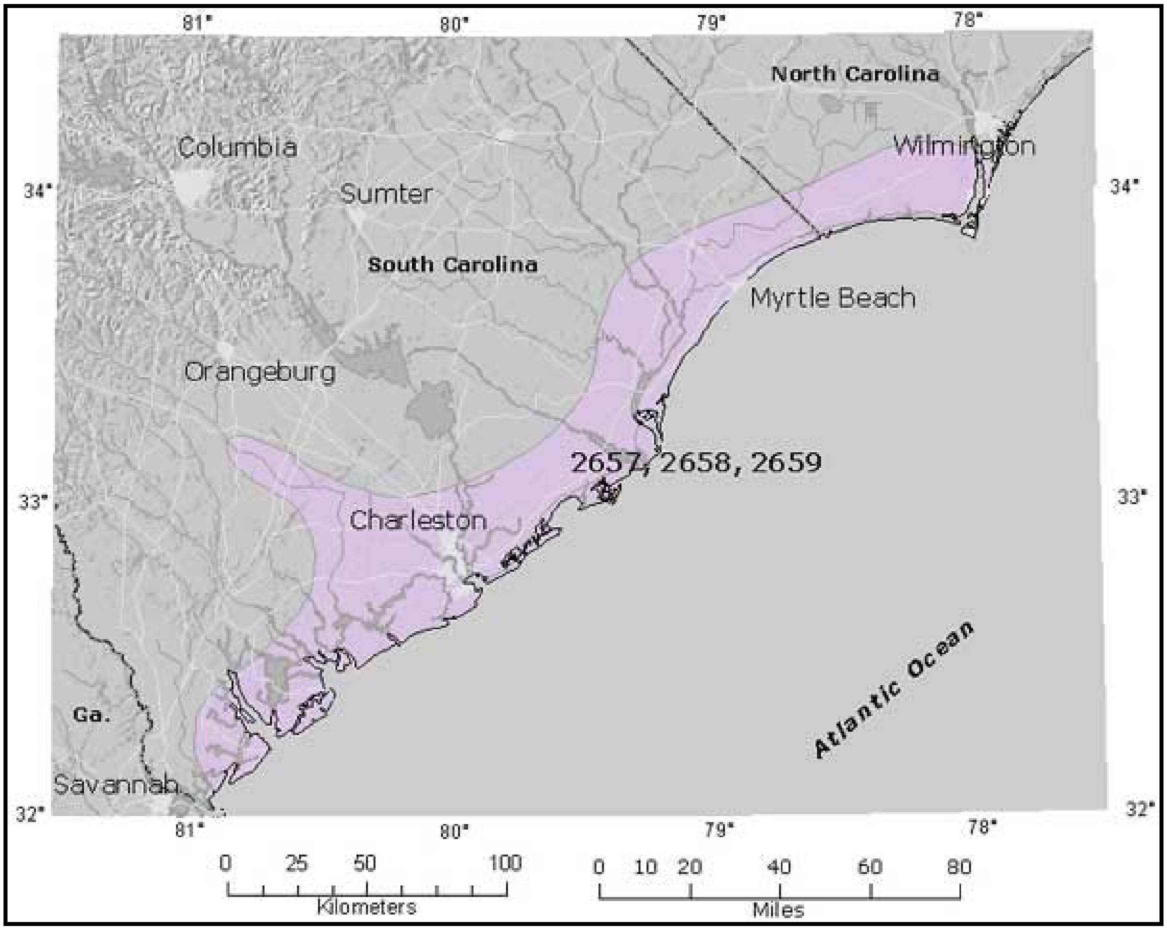


Figure 13-9, SC Quaternary Liquefaction Areas (USGS Website)

A database of extensive liquefaction studies at sites in China, Japan, California, and Alaska has been collected. Even though liquefaction has occurred in the CEUS, none of the liquefaction case histories have been evaluated, since most seismic events with earthquake magnitudes, M_w, greater than 6.5 occurred more than 100 years ago. Liquefaction evaluation in the CEUS and consequently in South Carolina, is relatively more complex than in other areas where liquefaction studies have been made because of the deep vertical soil column (up to 4,000 feet) encountered in the Atlantic Coastal plain, lack of recorded large seismic events, and uncertainty of the mechanisms and subsequent motions resulting from intraplate earthquakes (Schneider and Mayne, 1999). Never the less, historical soil liquefaction studies in the CEUS (Schneider and Mayne, 1999) indicate that current methods to evaluate cyclic liquefaction are in general agreement with predictions of cyclic liquefaction.

13.6 SOIL SHEAR STRENGTH LOSS SUSCEPTIBILITY SCREENING CRITERIA

Screening criteria is based on laboratory testing observations and on site parameters that have been observed to be present when soil SSL occurred in seismic hazard case histories. It has been observed that cyclic liquefaction potential decreases as soils increase in fines content (FC), increase in plasticity index (PI), and decrease in moisture content below the liquid limit (LL).

Screening for earthquake-induced soil SSL have traditionally been focused on cyclic liquefaction of cohesionless soils. Recent studies (Seed et. al, 2003, Boulanger and Idriss, 2004a, 2007; Bray and Sancio, 2006; Idriss and Boulanger, 2008) have stressed the need to evaluate loss in soil shear strength and stiffness due to cyclic liquefaction (low plasticity silts and clays) and cyclic softening (clays and plastic silts).

Seed et. al (2003) proposed the liquefaction susceptibility chart for fine grained soils shown in Figure 13-10 that is based on soil plasticity. The chart is divided into three zones of varying soil SSL susceptibility. Zone A has the highest potential for loss in shear strength resulting from cyclic liquefaction. Zone B was considered a transition area where soils could be subject to soil SSL and would require laboratory cyclic load testing for confirmation of soil shear strength susceptibility. Soils located in Zone C (Zone not covered by Zones A or B) were not susceptible to cyclic liquefaction induced soil SSL but were to be screened for soil SSL due to cyclic softening of sensitive cohesive soils.

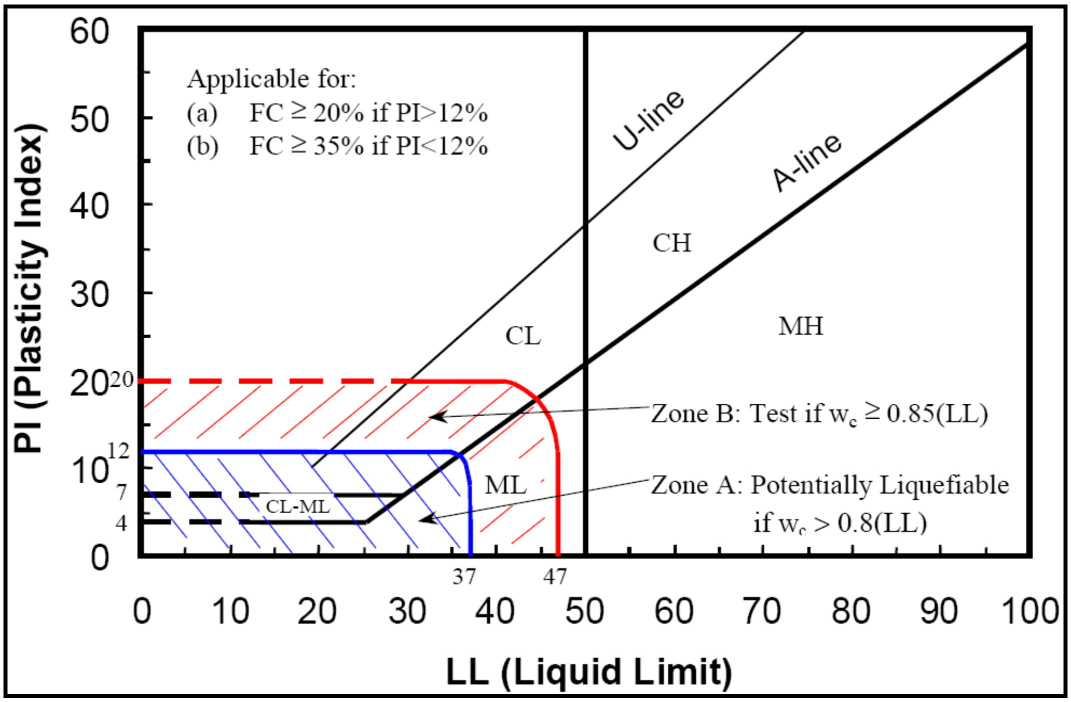


Figure 13-10, Liquefaction Susceptibility Based on Soil Plasticity (Seed, et al. 2003)

The liquefaction guidelines described by Seed et al. (2003) are best considered as envelopes of fine-grained soils that have been observed to experience significant strains or strength loss during earthquakes (Boulanger and Idriss, 2004a, 2006, 2007). Boulanger and Idriss (2004a, 2006, 2007) recommend that the fine-grained cyclic soil behavior would be best described as either Sand-Like or Clay-Like based on the Plasticity Index (PI). Boulanger and Idriss (2004a, 2006, 2007) suggested that there is a narrow soil SSL behavior transition zone between Sand-Like and Clay-Like that ranges from about a PI of 3 to 8 as indicated in Figure 13-11.

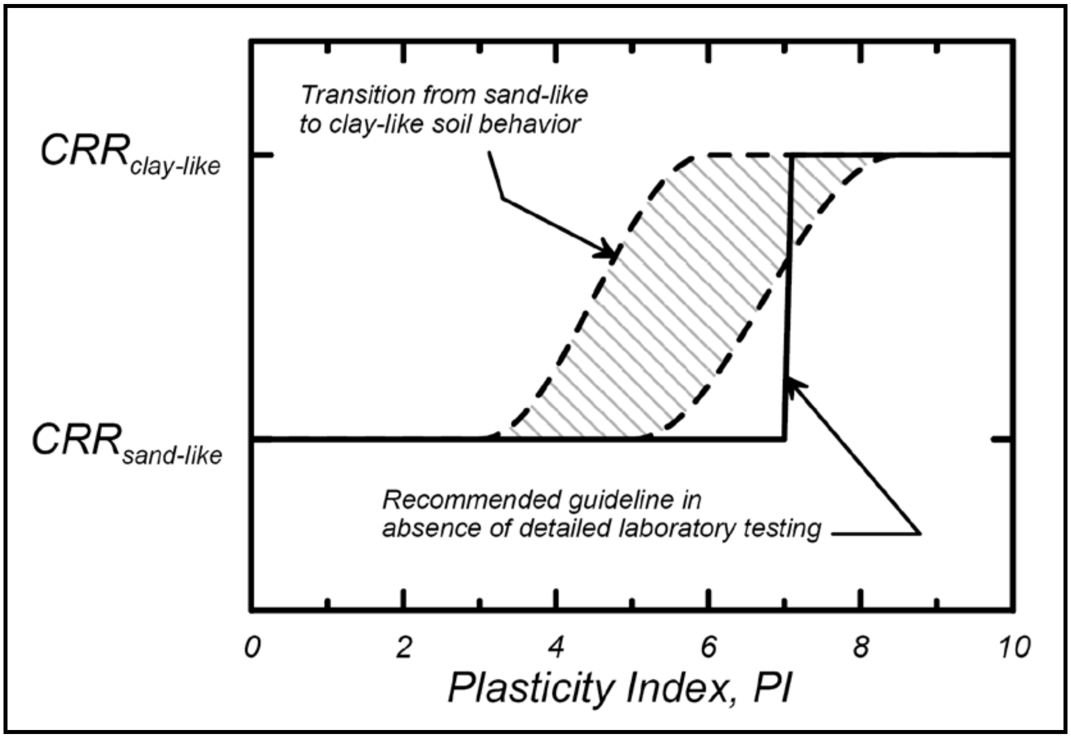


Figure 13-11, Transition from Sand-Like to Clay-Like behavior (Boulanger and Idriss, 2004a, 2006, 2007; Idriss and Boulanger, 2008) (With Permission from ASCE)

The soil SSL susceptibility screening criteria presented by Idriss and Boulanger (2008) for determining if the underlying subsurface soils at a project site are susceptible to soil SSL due to cyclic liquefaction or cyclic softening will be used. The soil SSL behavior screening adopted by SCDOT in the following Sections is consistent with Idriss and Boulanger (2008) and has been expanded to distinguish between normally and highly sensitive clays as indicated below. The soil SSL susceptibility criteria shall be based on the following three categories: (1) Sand-Like Soils (2) Normally Sensitive (NS) Clay-Like Soils, and (3) Highly Sensitive (HS) Clay-Like Soils.

Laboratory cyclic load testing of Sand-Like or Clay-Like soils is typically not required for typical bridges or typical transportation structures but may be required by the PCS/GDS on a project specific basis depending on the risk associated with the geotechnical seismic hazards under evaluation.

13.6.1 Sand-Like Soil

Soil SSL in Sand-Like soils is caused by cyclic liquefaction as described in Section 13.5.1. Sand-Like soils will be screened to a minimum depth of 80 feet below the existing ground surface or 20 feet beyond the lowest deep foundation element; whichever extent of screening is deeper.

Sand-Like soils that are susceptible to cyclic liquefaction are characterized by the following site and soil parameters:

1. Sand-Like soils susceptible to cyclic liquefaction must be below the water table. The water table selection for this evaluation must take into account the seasonal fluctuation of the ground water and the historic and/or possible future rise of the ground water level with respect to the soils being analyzed for liquefaction susceptibility. To determine the location of soils that are adequately saturated for liquefaction to occur, seasonally averaged groundwater elevations should be used. The Natural Resources Conservation Service (NRCS) website (http://websoilsurvey.nrcs.usda.gov/app/) may be consulted for determining the seasonal fluctuation of groundwater. Groundwater fluctuations caused by tidal action or seasonal variations will cause the soil to be saturated only during a limited period of time, significantly reducing the risk that liquefaction could occur within the zone of liquefaction.

2. Low plasticity fine grained soils that classify as CL-ML in the Unified Soil Classification System (USCS) with a Plasticity Index, $PI < 5$ (measured on soil portion passing the No. 40 sieve) and for all other fine grained soils with a Plasticity Index, $PI < 7$.

3. Corrected Standard Penetration Test (SPT) blow counts, $N^*_{1,60,CS} < 30$ blows/foot or normalized corrected Cone Penetration Test (CPT) tip resistance, $q_{C,1,N,CS} < 170$.

13.6.2 Normally Sensitive (NS) Clay-Like Soil

Soil SSL in NS Clay-Like soils is caused by cyclic softening as described in Section 13.5.2. Clay-Like soils will be screened to a minimum depth of 80 feet below the existing ground surface or 20 feet beyond the lowest deep foundation element; whichever extent of screening is deeper.

NS Clay-Like soils that are susceptible to cyclic softening are characterized by the following site soil parameters:

1. Fine grained soils that classify as CL-ML in the USCS with a Plasticity Index, $PI \geq 5$ (measured on soil portion passing the No. 40 sieve) and for all other fine grained soils with a Plasticity Index, $PI \geq 7$.

2. Soil sensitivity, $S_t < 5$ (Chapter 7)

13.6.3 Highly Sensitive (HS) Clay-Like Soil

Soil SSL in HS Clay-Like soils is caused by cyclic softening as described in Section 13.5.2. Clay-Like soils will be screened to a minimum depth of 80 feet below the existing ground surface or 20 feet beyond the lowest deep foundation element; whichever extent of screening is deeper.

HS Clay-Like soils that are susceptible to cyclic softening are characterized by the following site and soil parameters:

1. Fine grained soils that classify as CL-ML in the USCS the Plasticity Index, PI $\geq$ 5 (measured on soil portion passing the No. 40 sieve) and for all other fine grained soils the Plasticity Index, PI $\geq$ 7.
2. Soil sensitivity, $S_t \geq 5$ (Chapter 7)

13.7 SOIL SHEAR STRENGTH LOSS TRIGGERING FOR LEVEL GROUND SITES

The liquefaction triggering analyses for level ground sites will include an evaluation of Sand-Like and Clay-Like soils that were identified to be susceptible to cyclic liquefaction or cyclic softening during the screening process described in Section 13.6. A level ground site condition is defined as either level ground or gently sloping ground (slope angle < 5 degrees), or level/gently sloping ground with a nearby (≤ 150 feet) steep slope or free-face slope that is less than 16 feet in height. This site condition is typically encountered in the South Carolina Coastal Plain under roadways, at grade bridge crossings over non-navigable rivers or small creeks, floodplain crossings, etc. These level ground site conditions typically have very little, if any, initial static shear stresses in the underlying soils and therefore the effects of static shear stresses are typically not taken into account during liquefaction triggering analyses for level ground sites.

The *Simplified Procedures* for determining liquefaction triggering of Sand-Like soils shall be based on SPT in-situ testing or on CPT in-situ testing using the methods described in the Earthquake Engineering Research Institute (EERI) Monograph MNO-12, Soil Liquefaction During Earthquakes (Idriss and Boulanger, 2008).

Alternate methods of evaluating liquefaction triggering of Sand-Like soils such as those described in the 1996 NCEER and 1998 NCEER/NSF workshop (Youd, et al. 2001) may be required on a project specific basis.

The *Simplified Procedure* for determination of cyclic liquefaction triggering is an empirical method based on field investigations of sites that have or have not experienced liquefaction of Sand-Like soils. The *Simplified Procedure* for Sand-Like soils cannot differentiate between the types of liquefaction (flow liquefaction or cyclic softening). The *Simplified Procedure* for determining the onset/triggering of cyclic softening of Clay-Like soils during earthquake events is based on laboratory investigations. The PCS/GDS may require on a project specific basis, more rigorous analytical methods such as non-linear effective stress site response methods and advanced laboratory testing not included in this Manual.

The *Simplified Procedure* compares the earthquake-induced stresses (Demand, D), expressed in terms of an equivalent uniform cyclic stress ratio (CSR^*_{eq}) that has been magnitude-weighted ($CSR^*_{eq} = CSR_{M=7.5}$), with the capacity of the soil's resistance to soil SSL (Capacity, C), expressed in terms of corrected cyclic resistance ratio (CRR^*_{eq}) that also has been magnitude-weighted and normalized to an effective overburden stress of 1 tsf ($CRR^*_{eq} = CRR_{M=7.5,1\ tsf}$). The ratio of the earthquake-induced stresses (Demand, D) divided by the soils resistance to soil SSL (Capacity, C) defines the strength loss ratio $(D/C)_{SL}$. The LRFD equation that is to be used to evaluate the onset of strength loss (SL) at level ground site conditions is provided below:

$$\left(\frac{D}{C}\right)_{SL} = \frac{CSR_{eq}^{*}}{CRR_{eq}^{*}} \le \varphi_{SL} \qquad \text{Equation 13-1}$$

The onset of cyclic liquefaction (Sand-Like soils) or cyclic softening (Clay-Like soils) occurs when the strength loss ratio $(D/C)_{SL}$ is greater than the strength loss resistance factor (φ_{SL}) provided in Chapter 9 - Geotechnical Resistance Factors.

Since the *Simplified Procedure* is a deterministic procedure, a load factor, γ, of unity (1.0) is used and the resistance factor, φ, accounts for the site variability and the level of acceptable risk to triggering soil SSL. As research advances and soil SSL analytical models are calibrated for LRFD design methodology, adjustments will be made in the implementation of the LRFD design methodology.

The overall process for conducting a soil SSL triggering analysis using the *Simplified Procedure* for level project site conditions is presented in a flow chart in Figure 13-12. The analytical procedures for computing cyclic stress ratio (CSR) and cyclic resistance ratio (CRR) of Sand-Like soils and Clay-like soils are provided in Section 13.10 and Section 13.11, respectively.

Soils that are susceptible to cyclic liquefaction or cyclic softening will require additional analyses to evaluate the effects of the soil shear strength degradation as discussed in Section 13.12. Project sites that have subsurface soils with the potential for soil SSL will require the evaluation for soil SSL-induced geotechnical seismic hazards such as flow slide failure, lateral spread, and soil settlements. The analytical procedures to determine the magnitude and extent of these soil SSL-induced hazards are provided in this Chapter.

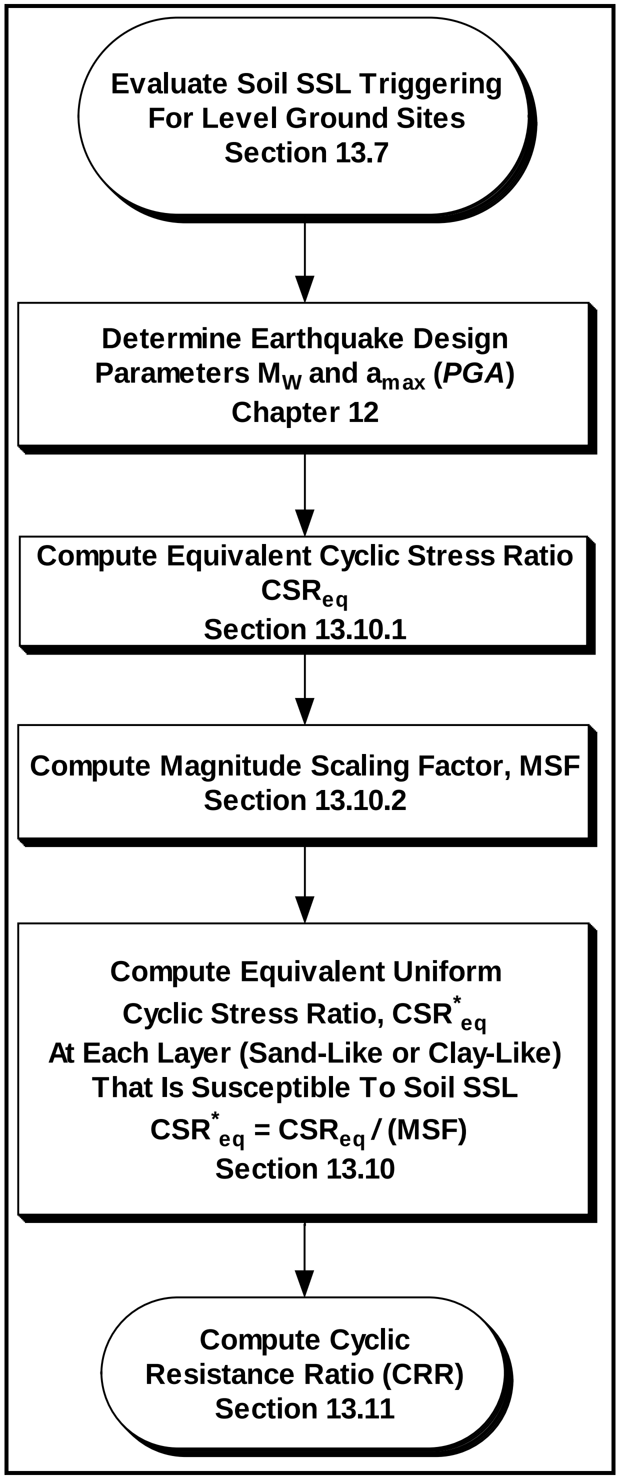


Figure 13-12, Soil SSL Triggering Analysis for Level Ground Sites

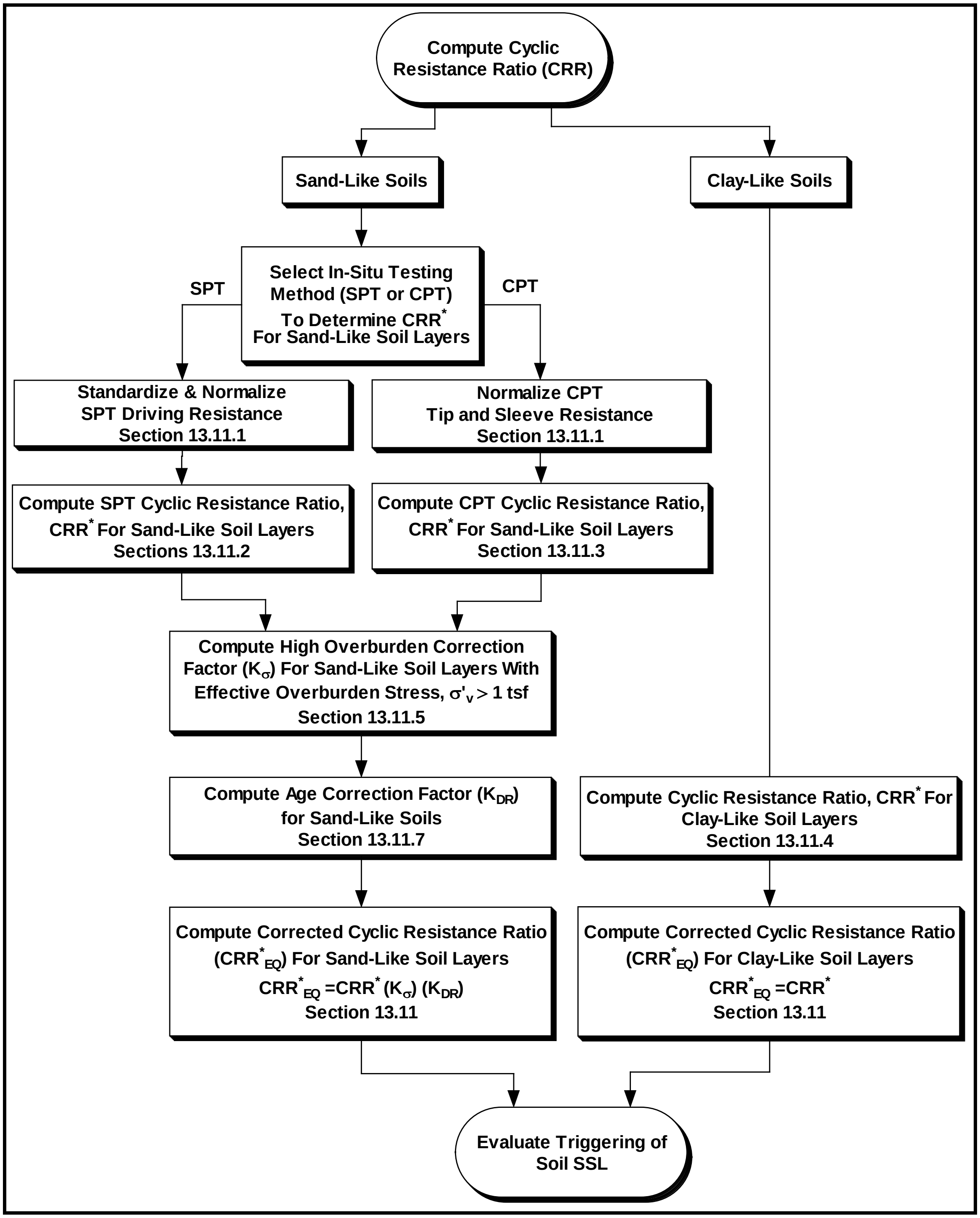


Figure 13-12 (Continued), Soil SSL Triggering Analysis for Level Ground Sites

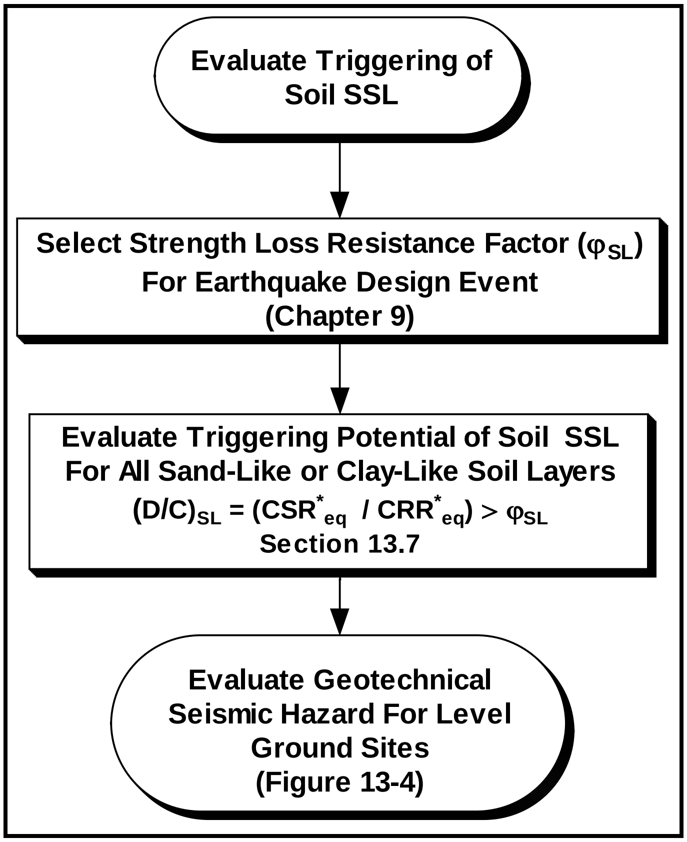


Figure 13-12 (Continued), Soil SSL Triggering Analysis for Level Ground Sites

13.8 FLOW FAILURE SCREENING FOR STEEPLY SLOPED GROUND SITES

Steeply sloped ground sites with soils susceptible to SSl (Section 13.6) will require a screening to determine if there is a high potential for flow failure resulting from soil SSL in Sand-Like and/or Clay-Like soils. Flow failure screening is evaluated by performing a post-seismic-strength-loss (PSSL) slope stability analysis.

In a PSSL slope stability analysis, Sand-Like soils are initially assumed to have cyclic liquefaction residual shear strength (τ_{rl}), NS Clay-Like soils are assumed to have cyclic softening residual shear strength (τ_{rs}), and HS Clay-Like soils are assumed to have remolded shear strength ($\tau_{remolded}$). The PSSL slope stability analysis shall be performed using Spencer's Slope Stability method in accordance with Chapter 17.

The PSSL slope stability is an evaluation of flow failure that is based on the evaluation of the ratio of driving forces (Demand, D) divided by resisting forces (Capacity, C) resulting in a flow failure ratio $(D/C)_{Flow}$. The LRFD equation that is used to evaluate the onset of flow failure is defined by the following equation.

$$\left(\frac{D}{C}\right)_{Flow} \leq \varphi_{Flow} \qquad \text{Equation 13-2}$$

The onset of flow failure occurs when the flow failure ratio $(D/C)_{Flow}$ is greater than the flow failure resistance factor (φ_{Flow}) provided in Chapter 9 - Geotechnical Resistance Factors.

Since slope instability analyses is a deterministic procedure, a load factor, γ, of unity (1.0) is used and the resistance factor, φ, accounts for the site variability and the level of acceptable risk to trigger flow failure. As research advances and flow failure analytical models are calibrated for LRFD design methodology, adjustments will be made in the implementation of the LRFD design methodology.

If the results of the PSSL slope instability analyses indicate the potential for flow failure; then Sand-Like soils and Clay-Like soils must be evaluated for triggering of soil SSL at steeply sloped ground sites. If the potential for flow failure does not exist; then it can be inferred that soil SSL will not occur and an evaluation of geotechnical seismic hazard for steeply sloped ground sites can proceed. The overall evaluation process for screening for flow failure due to PSSL in steeply sloped ground sites is shown in Figure 13-13.

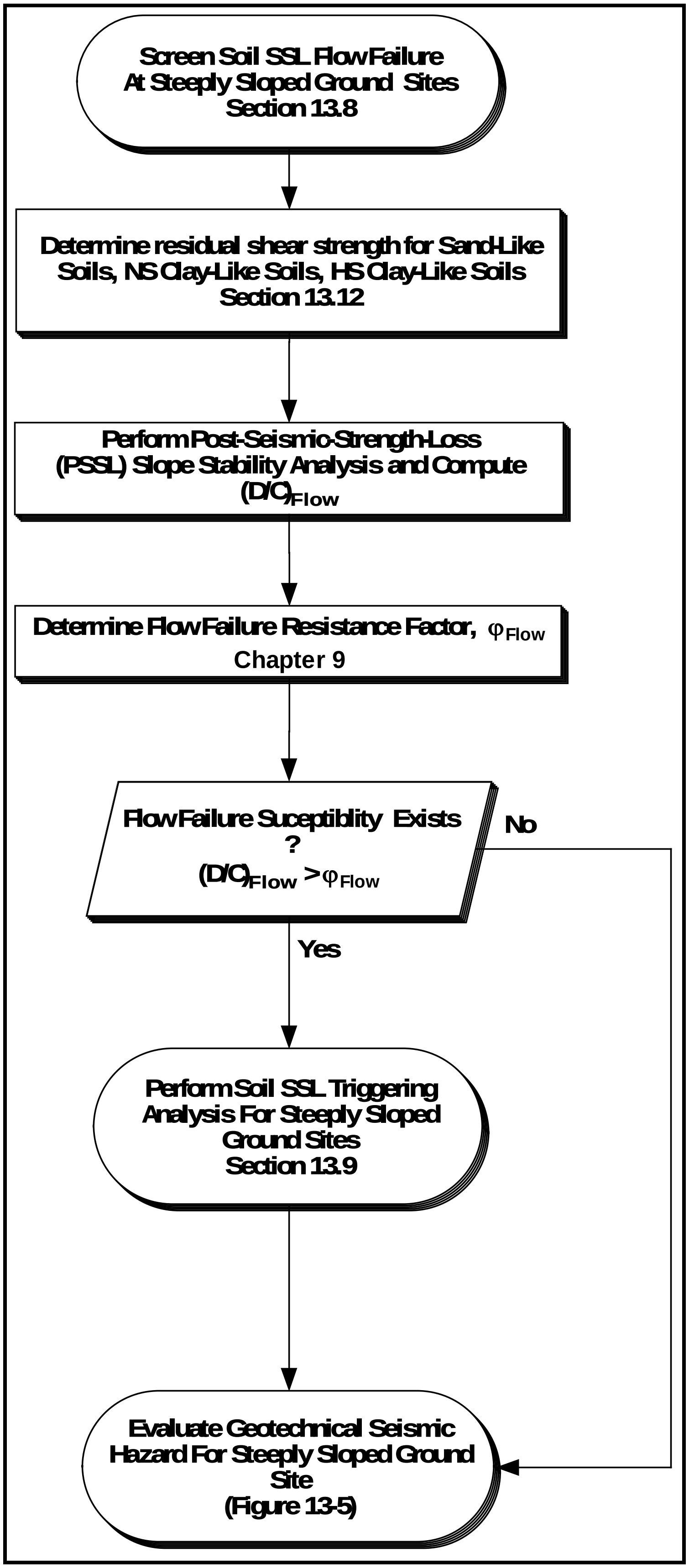


Figure 13-13, Flow Failure Screening - Steeply Sloped Sites

13.9 SOIL SHEAR STRENGTH LOSS TRIGGERING FOR STEEPLY SLOPED GROUND SITES

This Section describes the soil SSL triggering analysis for steeply sloped ground sites. A steeply sloped ground site is defined as a project site with a ground slope angle ≥ 5 degrees or a level ground site with a nearby (≤ 150 feet) steep slope or free-face slope that is ≥ 16 feet in height. This site condition is typically encountered at roadway embankments, cut hillsides, and sites where ERSs are constructed. The sloping ground conditions and any surcharges or surface loads will induce static shear stresses in the underlying soils that must be accounted for when evaluating liquefaction triggering for both Sand-Like and Clay-Like soils. The earthquake-induced stresses plus the static shear stresses (Demand, D) could potentially exceed the soil's resistance to soil SSL (Capacity, C) which would result in a reduction in soil shear strength.

The effects of static shear stress must be included in the evaluation of soils SSL triggering for steeply sloped site conditions by the methods indicated below:

1. **Static Shear Stress Ratio Correction Factor, K_α, Method:** The static shear stress ratio (SSSR) correction factor (K_α) method (Section 13.11.7) is presented by Idriss and Boulanger (2008) to account for static shear stresses in the *Simplified Procedure* method of evaluating soil SSL triggering for steeply sloped ground sites. The SSSR correction factor, K_α, method is further explained in Section 13.9.1.

2. **Shear Strength Ratio Method:** The shear strength ratio (SSR) triggering method computes the ratio of shear stress demand on the soil layer susceptible to soil SSL with the soil's yield strength. This method, developed by Olson and Stark (2003), uses the yield shear strength ratio and soil SSL ratio to evaluate the triggering of soil SSL for steeply sloped ground sites. The SSR method is further explained in Section 13.9.2.

Both methods presented above should be used to evaluate soil SSL triggering evaluation for steeply sloped ground sites, when the initial static stress ratio (α) is less than or equal to 0.35, and the results of each should be compared. When the maximum initial static stress ratio (α) is greater than 0.35, or when complex geometries and loadings need to be evaluated, the shear strength ratio (SSR) method presented in Section 13.9.2 should be used. Soils that are susceptible to cyclic liquefaction or cyclic softening will require additional analyses to evaluate the soil shear strength degradation (Section 13.12). Project sites that have subsurface soils with the potential for soil SSL will require the evaluation of soil SSL-induced geotechnical seismic hazards such as flow slide failure, seismic instability, and seismic soil settlement. The analytical procedures to determine the magnitude and extent of these soil SSL-induced hazards are provided in this Chapter.

13.9.1 Static Shear Stress Ratio Correction Factor, K_α, Method

The static shear stress ratio (SSSR) correction factor (K_α) can be used with the *Simplified Procedure* to evaluate the effects of initial static shear stresses for steeply sloped ground sites.

This is accomplished by multiplying the SSSR correction factor (K_α) by the soil's cyclic resistance ratio (CRR^*) as indicated in Section 13.11. The SSSR correction factor (K_α) proposed by Idriss and Boulanger (2008) is computed as indicated in Section 13.11.7. The SSSR correction factor (K_α) method is limited to a maximum initial static stress ratio α less than or equal to 0.35. The soil SSL triggering process for steeply sloped ground sites is shown in Figure 13-14.

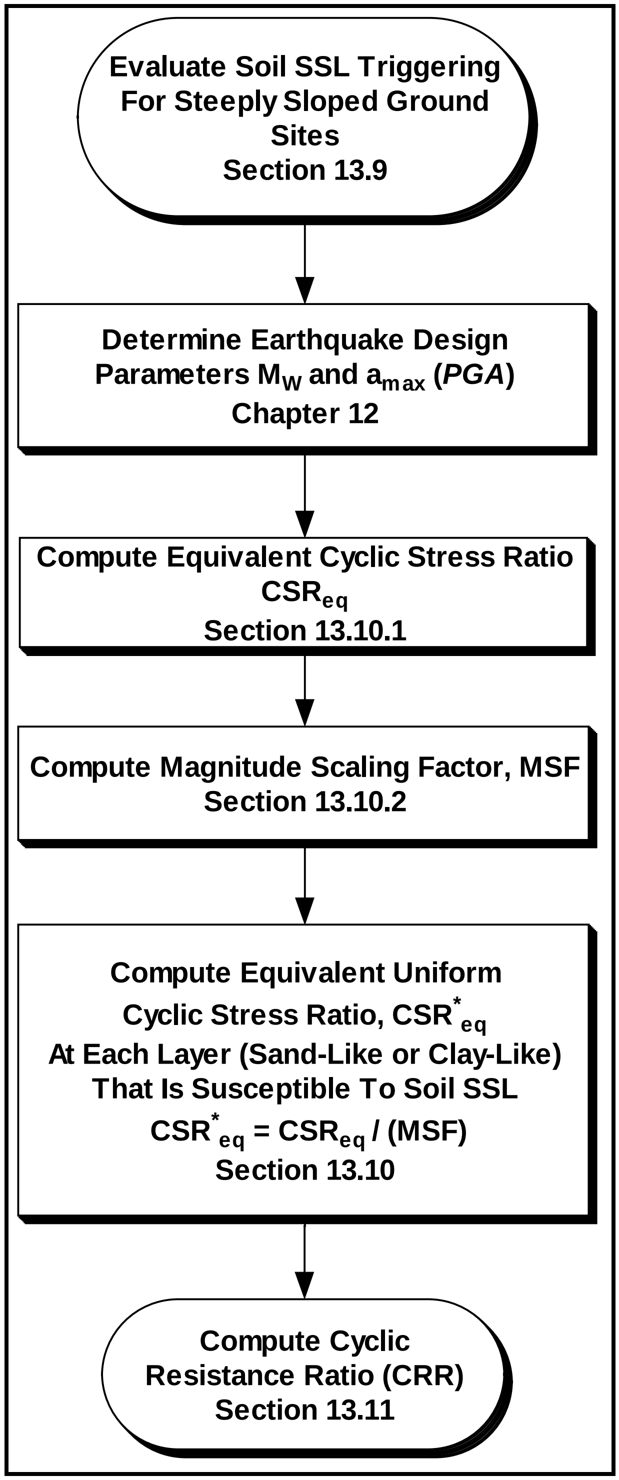


Figure 13-14, Simplified Procedure - Soil SSL At Steeply Sloped Ground Sites

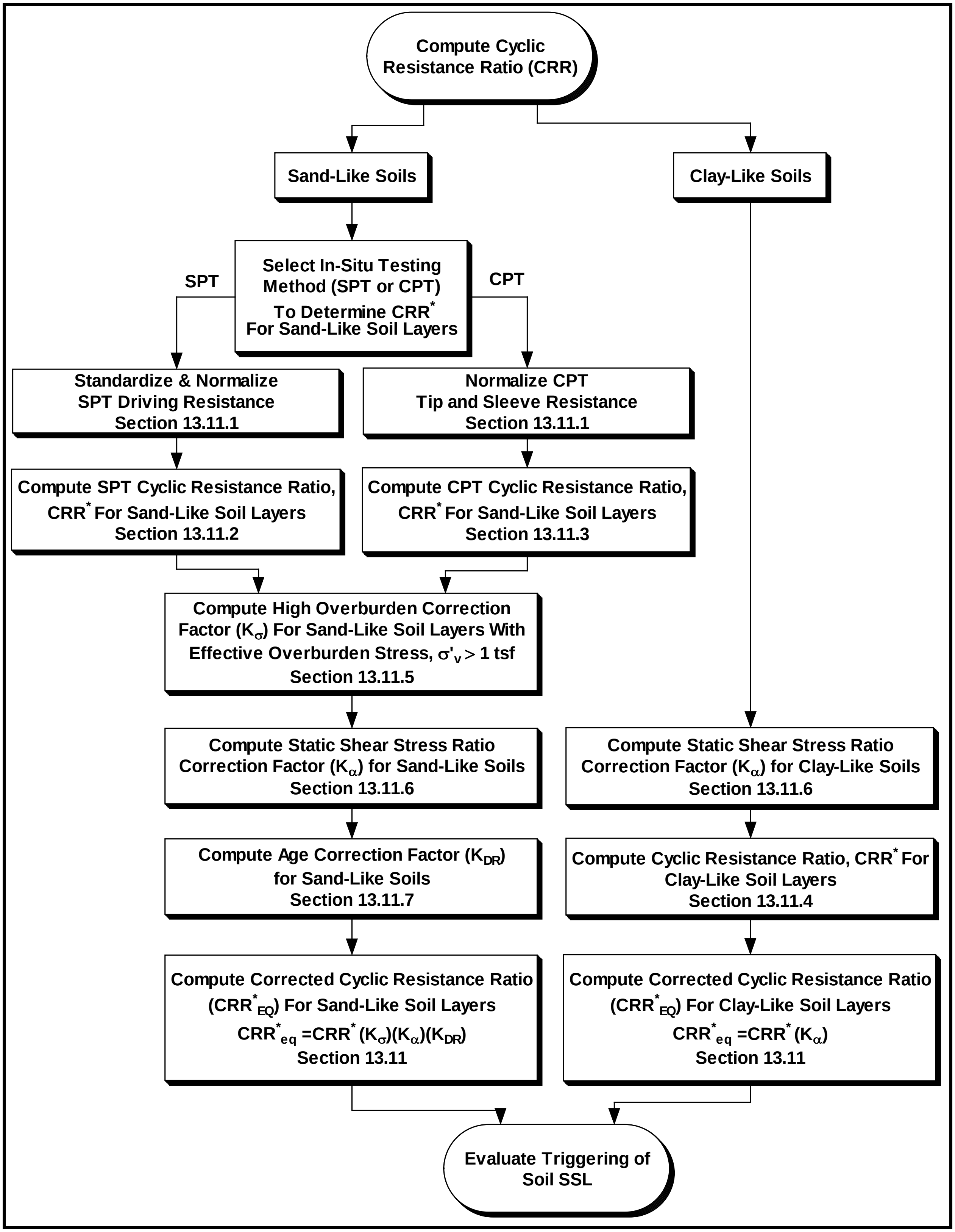


Figure 13-14 (Continued), Simplified Procedure - Soil SSL At Steeply Sloped Ground Sites

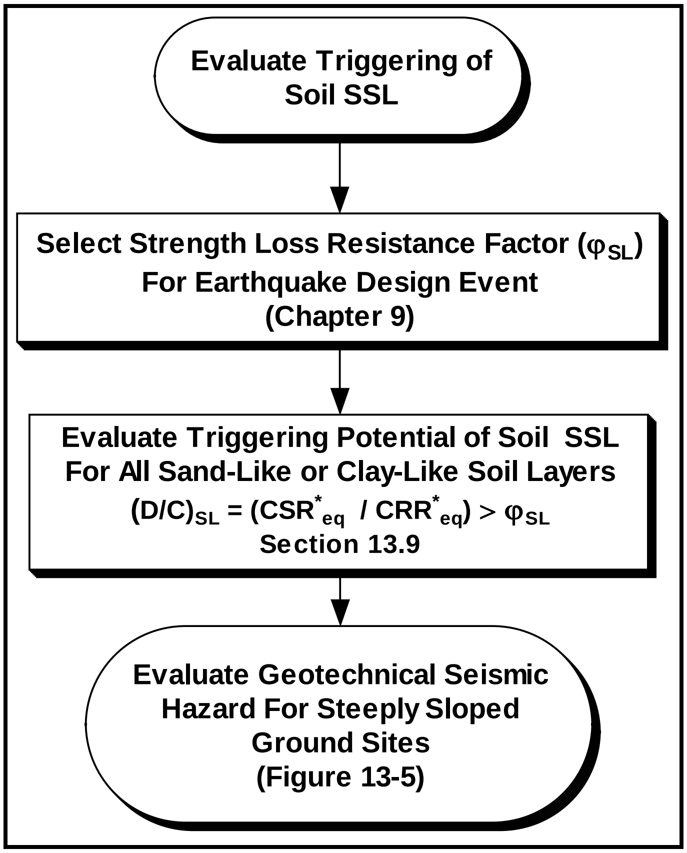


Figure 13-14 (Continued), Simplified Procedure - Soil SSL At Steeply Sloped Ground Sites

13.9.2 Shear Strength Ratio Triggering Method

The Shear Strength Ratio (SSR) method proposed by Olson and Stark (2003) has been adapted to evaluate cyclic liquefaction triggering for Sand-Like soils and cyclic softening of Clay-Like soils at steeply sloped sites subject to static shear stresses. This soil SSL triggering method consists of the following two parts:

1. Screen Sand-Like soils for Contractive behavior based on Contractive/Dilative correlations with in-situ testing (SPT and CPT) for Sand-Like soils (Section 13.9.2.1).

2. Evaluate soil SSL triggering of Sand-Like and Clay-Like soils by dividing the static (Section 13.9.2.3), seismic, and other shear stresses that the soil is exposed to (Demand, D) by the undrained shear strength of the soil (Capacity, C) to obtain the SSL ratio $(D/C)_{SL}$ and determine if the soil SSL triggering potential exists. The overall procedure is presented in Section 13.9.2.2.

13.9.2.1 Screening of Sand-Like Soils For Contractive Behavior

In addition to the soil SSL susceptibility screening criteria indicated in Section 13.6, this method requires the screening of Sand-Like soils for contractive behavior. Sand-Like soils must have contractive behavior in order to be subject to flow failure. The screening for contractive behavior is accomplished by plotting either SPT ($N^*_{1,60}$) or CPT ($q_{c,1}$) values on the horizontal axis as a function of the pre-failure vertical effective stress (σ'_{vo}) as indicated in Figure 13-15. After the in-situ testing values have been plotted, the Fear and Robertson (1995) soil boundary behavior relationship is plotted on the graph as indicated to determine which Sand-Like soil layers (Section 13.6) meet the contractive soil requirement of the SSR method. The Fear and Robertson (1995) soil boundary for contractive/dilative behavior relationship equations are provided below for CPT and SPT in-situ testing.

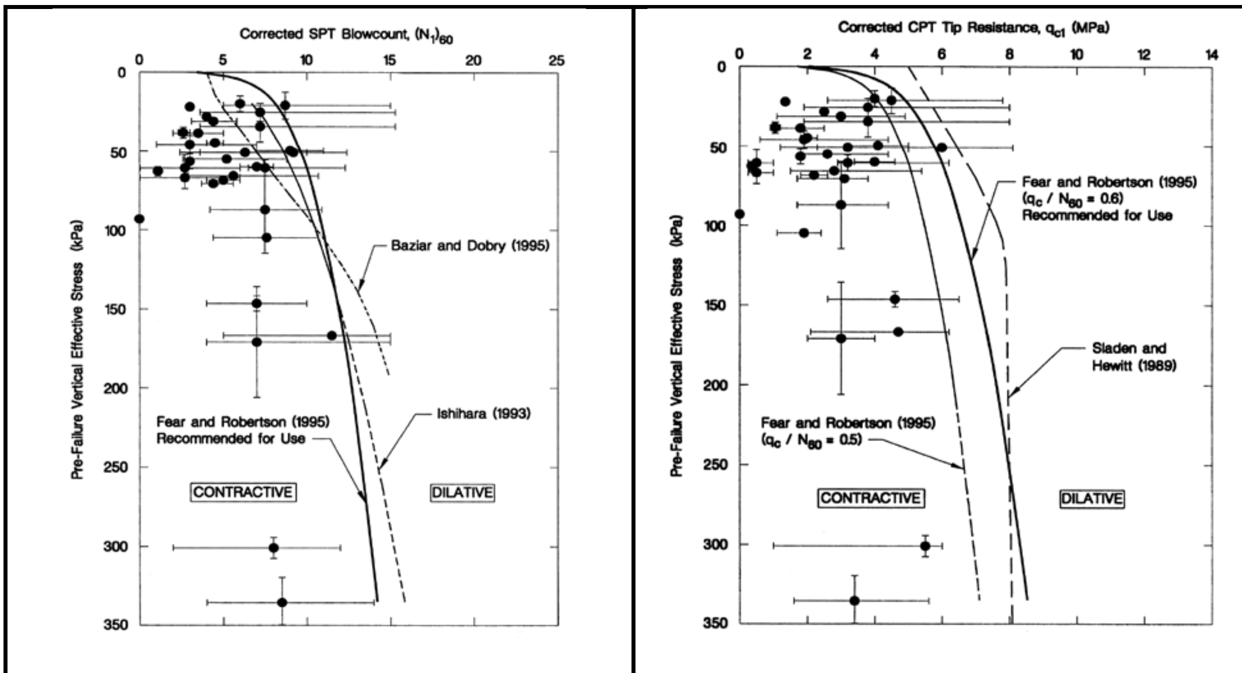
$$\left(\boldsymbol{\sigma}'_{vo}\right)_{SPT-Boundary} = 9.58 \times 10^{-4}\left(\boldsymbol{N}^*_{1,60}\right)^{4.79} \quad \text{Equation 13-3}$$

$$\left(\sigma'_{vo}\right)_{CPT-Boundary} = 1.10 \times 10^{-2}\left(\boldsymbol{q}_{c,1}\right)^{4.79} \quad \text{Equation 13-4}$$

Where,

- σ'_{vo} = Effective overburden stress (or σ'_v), units of kPa.
- $N^*_{1,60}$ = Normalized SPT-N values (Blows/foot) See Chapter Section 13.11.1 for SPT corrections.
- $q_{c,1}$ = CPT corrected tip resistance, units of MPA. See Section 13.11.1 for CPT corrections.

SPT or CPT values of Sand-Like soils that plot on the *Contractive* side of the boundary (left of boundary) are confirmed to be susceptible to flow failure as indicated by the liquefaction case histories evaluated by Olson and Stark (2003) plotted in Figure 13-15.



SPT ($N_{1,60}$) Contractive Behavior Evaluation	CPT ($q_{c,1}$) Contractive Behavior Evaluation

Figure 13-15, Contractive Soil Behavior Evaluation (Olson and Stark, 2003 with permission from ASCE)

13.9.2.2 Evaluate Soil SSL SSR Triggering Method

Soil SSL triggering of Sand-Like and Clay-Like soils is determined by dividing the static, seismic, and other shear stresses that the soil is subjected to (Demand, D) by the undrained shear strength of the soil (Capacity, C) to obtain the strength loss ratio $(D/C)_{SL}$. The LRFD equation that is used to evaluate the onset of strength loss (SL) at steeply sloped ground site conditions is provided below:

$$\left(\frac{D}{C}\right)_{SL} \leq \varphi_{SL} \qquad \text{Equation 13-5}$$

The Demand (D) is computed by adding the static driving shear stress (τ_{Static}), average seismic shear stress ($\tau_{Seismic}$), and other shear stresses (τ_{Other}) as indicated by the following equation.

$$D = \gamma\left(\tau_{Static} + \tau_{Seismic} + \tau_{Other}\right) \qquad \text{Equation 13-6}$$

The Capacity (C) of the soil is the undrained peak shear strength (τ_{Peak}) as determined for either Sand-Like Soils (Cohesionless) or Clay-Like Soils (Cohesive) as determined from Chapter 7. The peak undrained shear strength for cohesionless soils should be estimated based on the yield shear strength ($\tau_{Yield} = S_u(\text{yield})$) and the peak undrained shear strength ($\tau = \tau_{Yield}$) for cohesive soils should be estimated from either laboratory testing or in-situ testing.

The triggering of soil SSL for steeply sloped ground sites occurs when the strength loss ratio $(D/C)_{SL}$ is greater than the strength loss resistance factor (φ_{SL}) provided in Chapter 9.

$$\left(\frac{D}{C}\right)_{SL} = \frac{\left(\tau_{Static} + \tau_{Seismic} + \tau_{Other}\right)}{\tau_{Peak}} > \varphi_{SL} \qquad \text{Equation 13-7}$$

Since the SSR method for evaluating soil SSL triggering at steeply sloped project site conditions is a deterministic procedure, a load factor, γ, of unity (1.0) is used and the resistance factor, φ_{SL}, accounts for the site variability and the level of acceptable risk of soil SSL. As research advances and soil SSL analytical models are calibrated for LRFD design methodology, adjustments will be made in the implementation of the LRFD design methodology.

The process to evaluate triggering of soil SSL for steeply sloped ground is as follows:

1. The triggering of soil SSL begins by conducting a slope stability of the pre-failure geometry. The slope stability search should evaluate both circular and sliding wedge potential failure surfaces in accordance with Chapter 17. Spencer's Slope Stability method is required.

2. The critical failure surface is then divided into *n* slices (typically 10 to 15 slices) of length, L_i.

3. Compute the static shear stress (τ_{Static}) for each slope stability slice (length, L_i) susceptible to soil SSL at the onset of flow failure, in accordance with Section 13.9.2.3.

4. Compute the average, magnitude weighted, seismic induced stress ($\tau_{Seismic}$) for each slice (length, L_i) susceptible to soil SSL in accordance with the following equation.

$$\tau_{Seismic} = \frac{0.65 \times \tau_{max}}{MSF} \qquad \text{Equation 13-8}$$

Where,

τ_{max} = Maximum earthquake induced shear stress. τ_{max} is computed by the *Simplified Procedure* in accordance with the Section 13.10.1.1 or by performing a site-specific response analysis in accordance with Section 13.10.1.2.

MSF = Magnitude Scaling Factor computed in accordance with Section 13.10.2.

5. Compute any other shear stresses (τ_{Other}) that may be applicable such as those induced by surcharges, foundation loadings, etc.

6. Determine the value of the peak undrained shear strength ratio (τ_{Peak}/σ'_{vo} = $\tau_{Yield}/\sigma'_{vo}$) for Sand-Like soils (cohesionless soils) or the undrained shear strength ratio (τ_{Peak}/σ'_{vo} = S_u/σ'_{vo}) for Clay-Like soils (cohesive soils) in accordance with Chapter 7. Compute the undrained shear strength for Sand-Like soils (τ = τ_{Yield} = S_u(yield)) or Clay-Like soils (τ = S_u) for each slice of the critical failure surface by multiplying the peak undrained shear strength ratio (τ/σ'_{vo}) by the effective overburden stress (σ'_{vi}) for each slice.

7. Compute the soil SSL resistance ratio $(D/C)_{SL}$ as indicated by the following equation for each slice.

$$\left(\frac{D}{C}\right)_{SL} = \frac{\left(\tau_{Static} + \tau_{Seismic} + \tau_{Other}\right)}{\tau} \qquad \text{Equation 13-9}$$

8. The onset of cyclic liquefaction in Sand-Like soils or cyclic softening in Clay-Like soils, occurs when the strength loss ratio $(D/C)_{SL\text{-}i}$ for each slice (length, L_i) susceptible to soil SSL is greater than the LRFD resistance factor (φ_{SL}) presented in Chapter 9 as indicated by the following equation.

$$\left(\frac{D}{C}\right)_{SL-i} > \varphi_{SL} \qquad \text{Equation 13-10}$$

The overall process for conducting a soil SSL triggering analysis using the SSR method for steeply sloped project site conditions is presented in a flow chart in Figure 13-16.

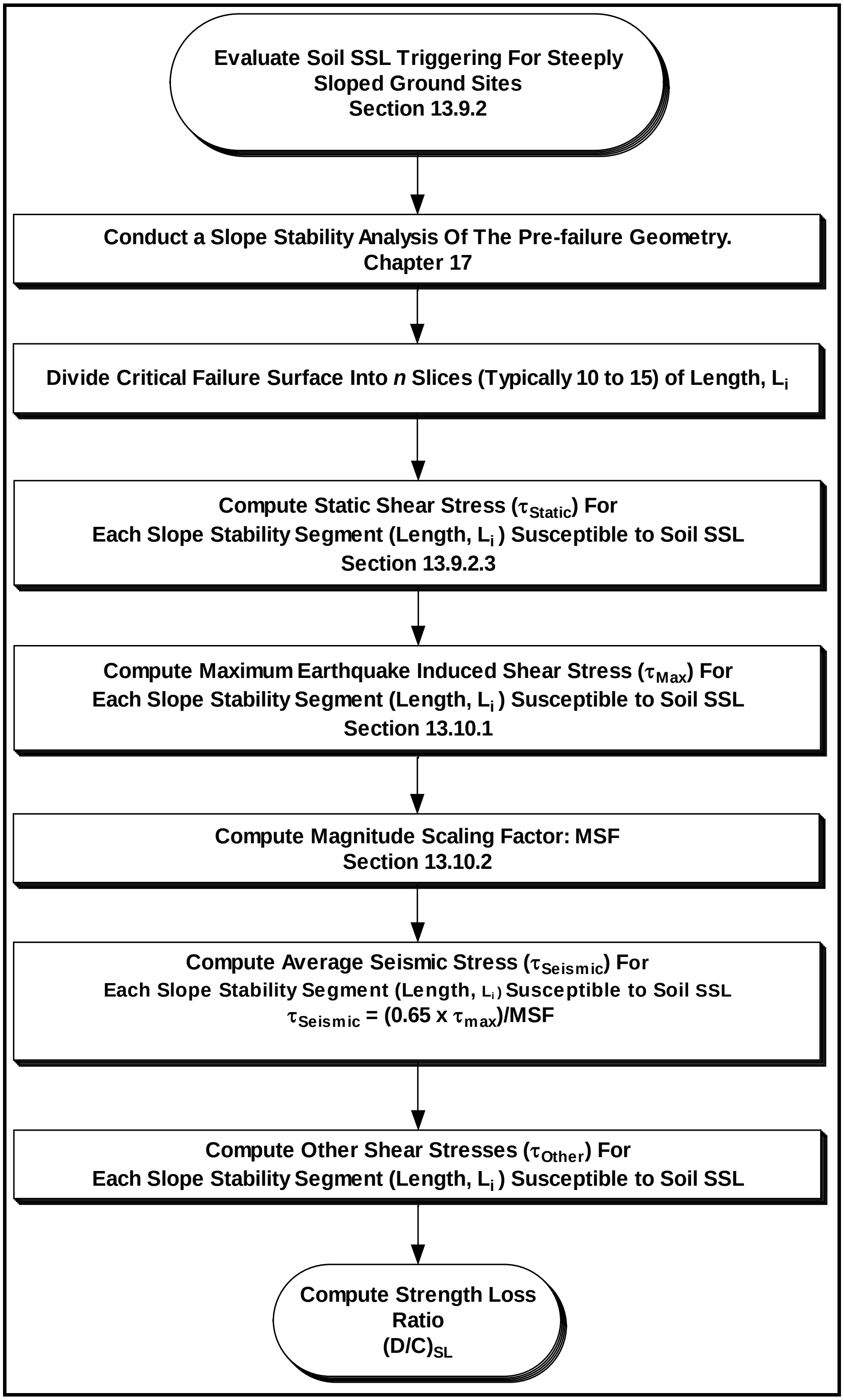


Figure 13-16, SSRA Soil SSL Triggering At Steeply Sloped Ground Sites

Compute Strength Loss Ratio $(D/C)_{SL}$

↓

Compute Total Stresses For Each Slope Stability Segment (Length L_i) Susceptible to Soil SSL
Demand = D = $(\tau_{Static} + \tau_{Seismic} + \tau_{Other})$

↓

Compute Peak Shear Strength (τ_{Peak}) to Resist Soil SSL For Each Slope Stability Segment (Length, L_i) Susceptible to Soil SSL
Capacity = C = τ_{Peak}
Chapter 7

↓

Compute Strength Loss Ratio $(D/C)_{SL}$ For Each Slope Stability Segment (Length, L_i) Susceptible to Soil SSL
$(D/C)_{SL} = (\tau_{Static} + \tau_{Seismic} + \tau_{Other}) / \tau_{Peak}$

↓

Select Soil Shear Strength Loss Resistance Factor (φ_{SL}) For Earthquake Design Event
Chapter 9

↓

Evaluate Triggering Potential For Each Slope Stability Segment (Length L_i) Susceptible to Soil SSL
$(D/C)_{SL} = (\tau_{Static} + \tau_{Seismic} + \tau_{Other}) / \tau_{Peak} > \varphi_{SL}$
Section 13.9

↓

Evaluate Geotechnical Seismic Hazard For Steeply Sloped Ground Sites (Figure 13-5)

Figure 13-16 (Continued), SSRA Soil SSL Triggering At Steeply Sloped Ground Sites

13.9.2.3 Static Shear Stress (τ_{Static}) of Soils Susceptible to Soil SSL

The static shear stress (τ_{Static}) at the onset of flow failure ($(D/C)_{Stability} = 1$) for each soil layer susceptible to soil SSL (Sand-Like soils and Clay-Like soils) can be computed by performing a slope stability back analysis of the pre-failure geometry. The slope stability search should evaluate both circular and sliding wedge potential failure surfaces in accordance with Chapter 17 with Spencer's Slope Stability method being required as well. Slope stability back analysis of the static shear stress (τ_{Static}) for a single soil layer susceptible to soil SSL is relatively straight forward when compared to slope failure surfaces that have multiple soil layers that are susceptible to soil SSL as indicated by the following procedure.

1. **<u>Single Layer of Soil SSL:</u>** The soil layers that intersect the failure surface that are not susceptible to SSL are assigned fully mobilized drained or undrained shear strengths. The soil layers susceptible to soil SSL that intersect the failure surface are initially assigned a "trial" soil shear strength (τ). The "trial" soil shear strength (τ) should range from the residual shear stress due to soil SSL (τ_{rl}, τ_{rs}, or $\tau_{remolded}$) to the fully mobilized undrained or drained shear strengths (τ or τ'). The soil shear strength (τ) for the layer susceptible to soil SSL is varied iteratively until the slope stability ratio, $(D/C)_{Stability} = 1$ that is equivalent to the static driving stress (τ_{Static}) prior to flow failure of the slope. Alternatively, some slope stability software allow the input of the static shear strength ratio directly ($\alpha = \tau_{Static}/\sigma'_{vo}$) for soil layers susceptible to soil SSL. For this software option, the static shear strength ratio (α) is varied iteratively until the slope stability ratio, $(D/C)_{Stability} = 1$ that is equivalent to the static shear strength ratio ($\alpha = \tau_{Static}/\sigma'_{vo}$) prior to failure of the slope.

2. **<u>Multiple Layers of Soil SSL:</u>** A comprehensive methodology for multiple layers of soils susceptible to soil SSL within the failure surface has not been fully developed as of this time. The following procedure can conservatively evaluate soil SSL triggering when multiple soil layers are susceptible to soil SSL.

   A. Determine the static shear stress (τ_{Static}) for each soil layer susceptible to soil SSL by assuming that each layer behaves independently of the other layers using the method described above in Item 1 for "Single Layer of Soil SSL." This assumption does not take into account any redistribution of static shear stresses between soil layers that are susceptible to soil SSL.

   B. Continue with the soil SSL triggering analysis.

   C. Each layer that has a shear loss ratio $(D/C)_{SL} > \varphi_{SL}$ shall be conservatively assumed to have the potential for soil SSL triggering.

D. Each layer that has a shear loss ratio $(D/C)_{SL} \leq \varphi_{SL}$, indicates that soil SSL triggering would not occur if it were a single layer of soil. Since there are multiple layers of soil SSL susceptibility this soil layer may still be subject to soil SSL as a result of static shear stress redistribution when soil SSL occurs in other layers within the failure surface. Another slope stability back analysis of the layer being evaluated to determine the static shear stress (τ_{Static}) for this layer is required using the method described above for "Single Layer of Soil SSL" and the following soil shear strength parameters. Soils not susceptible to soil SSL are assigned fully mobilized drained or undrained shear strengths. Soils that have been identified as having the potential for soil SSL are assigned residual shear stress due to soil SSL (τ_{rl}, τ_{rs}, or $\tau_{remolded}$) in accordance with Section 13.12. Continue with the soil SSL triggering analysis. If the soil layer being re-analyzed has a shear loss ratio $(D/C)_{SL} > \varphi_{SL}$, the layer shall be conservatively assumed to have the potential for soil SSL triggering. If the shear loss ratio $(D/C)_{SL} \leq \varphi_{SL}$, the layer shall be assumed not to be subject to soil SSL triggering.

An alternative to this procedure that is complex and requires significant experience would be to perform a numerical analysis of the slope stability to evaluate the static shear stresses (τ_{Static}) for the soil layers susceptible to soil SSL. Numerical modeling of the slope stability to obtain static stresses will require approval from the PCS/GDS. If numerical modeling is permitted, only the static driving shear stresses (τ_{Static}) or static shear strength ratio ($\alpha = \tau_{Static}/\sigma'_{vo}$) computed will be used for the evaluation of soil SSL triggering potential. The numerical modeling shall be compared to the method provided above.

3. If the static shear strength ratio is computed directly ($\alpha = \tau_{Static}/\sigma'_{vo}$) for soil layers susceptible to soil SSL, the static shear stress (τ_{Static}) for this layer can be computed by multiplying the static shear strength ratio (α) by the effective overburden stress, σ'_{vi}, for that segment represented along the soil failure surface.

4. If the soil shear strength (τ) that has been computed corresponds to the static driving stress (τ_{Static}) prior to failure of the slope, then divide the complete critical failure surface into n slices (typically 10 to 15 slices) of length, L_i.

5. Determine the weighted average effective vertical overburden stress (σ'_{vo}) as indicated by the following equation:

$$\overline{\sigma}'_{vo} = \frac{\sum_{i=1}^{n} \sigma'_{vi} L_i}{\sum_{i=1}^{n} L_i} \qquad \text{Equation 13-11}$$

Where σ'_{vi} = effective vertical overburden stress at each failure slice along the soil failure surface.

6. Calculate the average static shear stress ratio (τ_{Static} / $\overline{\sigma}'_{vo}$) for all soil layers susceptible to soil SSL.σ

7. Compute the static driving shear stress(τ_{Static}) for each slice of the critical failure surface (length, L_i) by multiplying the average static shear stress ratio (τ_{Static} / $\overline{\sigma}'_{vo}$) for that soil layer by the effective overburden stress for the slice, σ'_{vi}, along the soil failure surface.

8. The static shear strength ratio ($\alpha = \tau_{Static}$ / σ'_{vo}) for soil layers can be computed by dividing static driving shear stress (τ_{Static}) by the effective overburden stress for the slice, σ'_{vi}, along the soil failure surface.

13.10 CYCLIC STRESS RATIO (CSR)

The earthquake-induced cyclic stresses in the soil (Demand, D) are quantified by the cyclic stress ratio (CSR). The equivalent uniform cyclic stress ratio, CSR^*_{eq}, is the equivalent uniform earthquake-induced stress that has been magnitude-weighted (M_w = 7.5) as shown in the following equation:

$$CSR^*_{eq} = CSR_{eq,7.5} = \frac{CSR_{eq}}{(MSF)} \qquad \text{Equation 13-12}$$

Where,

CSR_{eq} = is the equivalent earthquake-induced stress (Section 13.10.1)
MSF = is the Magnitude Scaling Factor (Section 13.10.2)

13.10.1 Equivalent Earthquake-Induced Stress (CSR_{eq})

The equivalent earthquake-induced stress, CSR_{eq}, sometimes referred to as the average earthquake-induced stress, is defined as shown in the following equation:

$$CSR_{eq} = 0.65\, CSR_{Peak} \qquad \text{Equation 13-13}$$

Where CSR_{Peak} is the maximum earthquake-induced cyclic stress ratio. Note that a factor of 0.65 is included in Equation 13-13 to obtain an “average” or equivalent CSR_{eq} value. The method of computing the maximum earthquake-induced stress ratio, CSR_{Peak}, depends on the method of performing the site response analysis discussed in Chapter 12.

13.10.1.1 Simplified Procedure Determination of CSR_{Peak}

The *Simplified Procedure* for determination of the CSR_{Peak} should typically be used for evaluation of soil SSL. The *Simplified Procedure* for computing CSR_{Peak} is shown in the following equation:

$$CSR_{Peak} = \frac{\tau_{Max}}{\sigma'_{vo}} = \left(\frac{a_{\max}}{g}\right)\left(\frac{\sigma_v}{\sigma'_{vo}}\right) r_d \qquad \text{Equation 13-14}$$

Where,

a_{max} = Peak horizontal ground acceleration is in units of acceleration due to gravity (g) . The peak horizontal ground acceleration is determined from the three-point design response spectrum method in Chapter 12.

σ_v = Total overburden stress

σ'_{vo} = Effective overburden stress

r_d = Shear stress reduction coefficient (dimensionless)

τ_{max} = Maximum earthquake induced stress with depth. In the Simplified Procedure the maximum earthquake induced stress (τ_{max}) is approximated by the following equation.

$$\tau_{Max} = \left(\frac{a_{\max}}{g}\right)\sigma_v\ r_d \qquad \text{Equation 13-15}$$

The shear stress reduction coefficient, r_d, is a parameter that describes the ratio of cyclic stresses for a flexible column to the cyclic stresses of a rigid column ($r_d = \tau_{soil\ column} / \tau_{rigid}$). For an r_d = 1, the flexibility of the soil column would correspond to rigid body behavior. One-dimensional dynamic site response studies (Seed and Idriss, 1971: Golesorkhi, 1989; Idriss, 1999; and Cetin et al., 2004) have shown that the shear stress reduction factor is dependent on the ground motion characteristics (i.e. intensity and frequency content), shear wave velocity profile of the site (i.e. site stiffness), and nonlinear dynamic soil properties. Idriss (1999) performed several hundred parametric site response analyses and developed a shear stress reduction coefficient, r_d that was expressed as a function of depth and earthquake moment magnitude (M_W) as indicated in Figure 13-17.

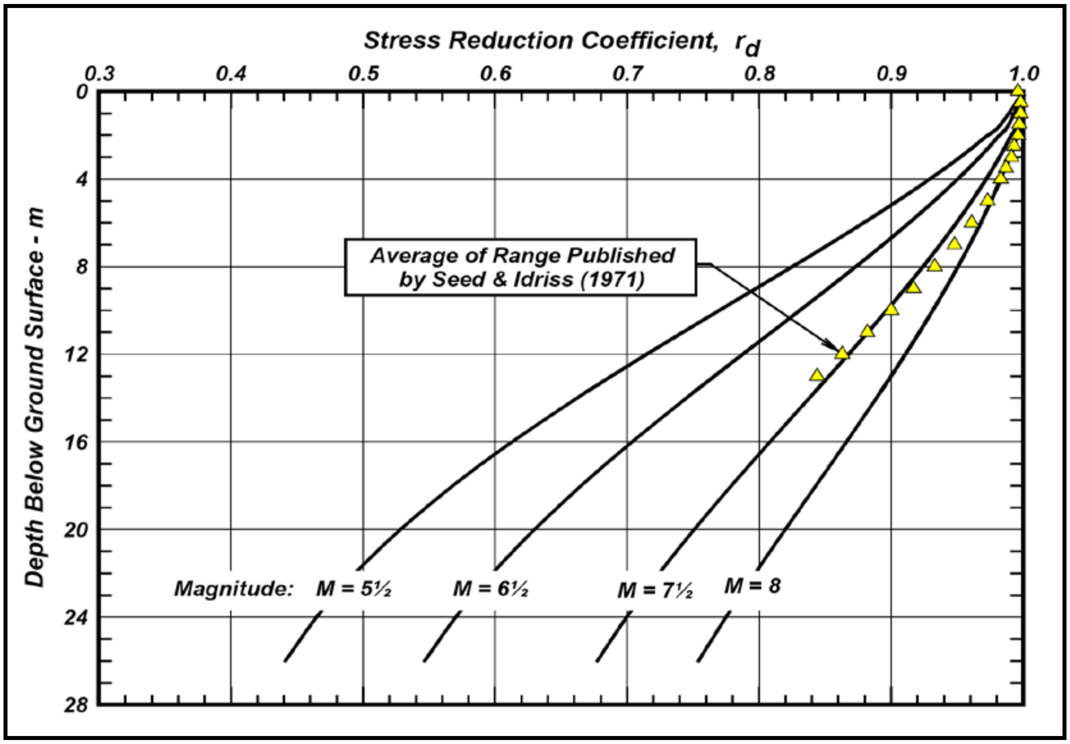


Figure 13-17, Variations of Shear Stress Reduction Coefficient, r_d (Idriss, 1999)

Shear stress reduction coefficient (r_d) equations for US customary units were modified from SI equations proposed by Idriss (1999) as indicated below.

$$\boldsymbol{r_d} = \mathbf{exp}\left(\alpha + \left(\beta \cdot \boldsymbol{M_w}\right)\right) \qquad \text{Equation 13-16}$$

$$\boldsymbol{\alpha} = -1.012 - 1.126\sin\left(\left(\frac{\left(\frac{\boldsymbol{z}}{3.28}\right)}{11.73}\right) + 5.133\right) \qquad \text{Equation 13-17}$$

$$\boldsymbol{\beta} = 0.106 + 0.118\sin\left(\left(\frac{\left(\frac{\boldsymbol{z}}{3.28}\right)}{11.28}\right) + 5.142\right) \qquad \text{Equation 13-18}$$

Where,

z	=	Depth below ground surface (feet)
M_W	=	Earthquake moment magnitude

Note that the arguments inside the "**sin**" terms above are in radians. For the purposes of evaluating soil SSL, the CSR_{Peak} should not be evaluated using this method for depths greater than 80 feet (24 m). The uncertainty increases for shear stress reduction coefficients (r_d) at depths greater than *z* > 65 feet (20 m). When the maximum earthquake-induced stress ratio,

CSR_{Peak}, is required for depths greater than 80 feet, a site-specific response analysis (Section 13.10.1.2) may be warranted with approval of the PCS/GDS.

13.10.1.2 Site Specific Response Determination of CSR_{Peak}

When approved by the PCS/GDS, the maximum earthquake-induced stress ratio, CSR_{Peak}, can be computed for depths greater than 80 feet (24 m) by using the results of a site-specific seismic response analysis (Chapter 12) as indicated by the following equation.

$$CSR_{Peak} = \frac{\tau_{\max}}{\sigma'_{vo}} \qquad \text{Equation 13-19}$$

Where τ_{max} is the maximum earthquake-induced cyclic shear stress that is obtained from the site-specific response analysis of the ground motions and σ'_{vo} is the effective overburden stress at the depth being evaluated. Site-specific seismic response analyses referenced in Chapter 12 are typically one-dimensional equivalent linear analyses. Because the one-dimensional equivalent linear analyses have a reduced reliability as ground shaking levels (PGA) increase above 0.40g in softer soils or where the maximum shearing strain amplitudes exceed 1 to 2 percent, a comparison with the *Simplified Procedure* should be performed for depths greater than 80 feet (24 m) and the more conservative values should be used. In lieu of using the more conservative analytical results, the PCS/GDS should be consulted to determine if a nonlinear effective stress site response method should be used to determine the maximum earthquake-induced shear stress, τ_{max}.

13.10.2 <u>Magnitude Scaling Factor (MSF)</u>

The Magnitude Scaling Factor, MSF, is used to scale the equivalent uniform earthquake-induced stresses, CSR_{eq}, to the duration that is typical of an average earthquake event of magnitude, $M_w = 7.5$. A large amount of scatter in the magnitude scaling factor, MSF, is observed from various studies presented in Youd et al. (2001), particularly at the lower range of earthquake moment magnitudes ($5.5 < M_w < 6.5$). Boulanger and Idriss (2007) have recommended MSF for Sand-Like soils (MSF_{Sand}) and for Clay-Like soils (MSF_{Clay}) as indicated in Figure 13-18. Because the predominant earthquake in South Carolina had an approximate earthquake magnitude of 7.3 and the target scaling earthquake is a 7.5, the variability observed in the magnitude scaling factor studies should have minimal impact on the liquefaction analyses.

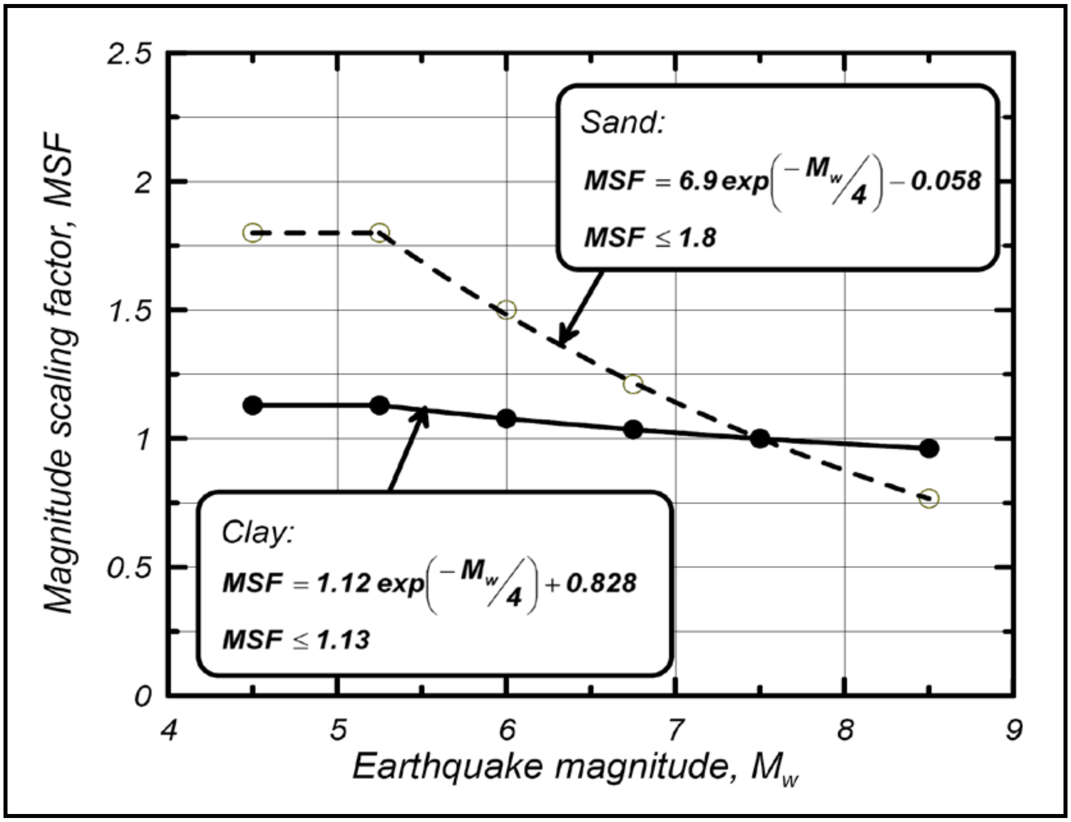


Figure 13-18, Magnitude Scaling Factor (MSF)
(Boulanger and Idriss, 2007 with permission from ASCE)

In lieu of using Figure 13-18, the following equations may be used to compute the MSF_{Sand} and MSF_{Clay}.

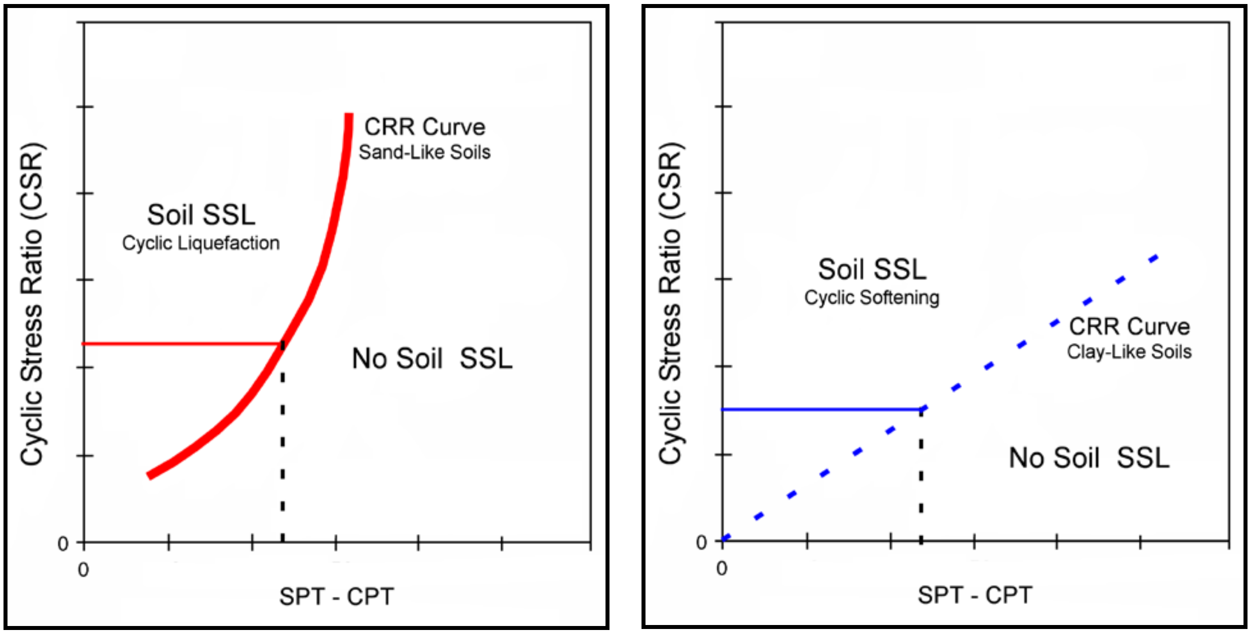
$$MSF_{Sand} = 6.9 \cdot \exp(-0.25 M_W) - 0.058 \leq 1.80 \qquad \text{Equation 13-20}$$

$$MSF_{Clay} = 1.12 \cdot \exp(-0.25 M_W) + 0.828 \leq 1.13 \qquad \text{Equation 13-21}$$

Where, M_w is the moment magnitude of the design earthquake being evaluated for soil SSL triggering.

13.11 CYCLIC RESISTANCE RATIO (CRR)

The soil's resistance to soil SSL (Capacity, C) is quantified by the cyclic resistance ratio (CRR). The cyclic resistance ratio (CRR) for Sand-Like soils is typically characterized as a curvilinear boundary that indicates the relationship between cyclic stress ratio (CSR) and in-situ testing results from SPT or CPT. The cyclic resistance ratio (CRR) for Clay-Like soils is typically characterized as a linear reduction of the undrained shear strength that indicates the relationship between cyclic stress ratio (CSR) and in-situ testing results from SPT or CPT. A typical cyclic resistance ratio (CRR) curve for Sand-Like soils is shown in Figure 13-19(A) and for Clay-Like soils is shown in Figure 13-19(B).



(A) CRR Curve Sand-Like Soils **(B) CRR Curve Clay-Like Soils**

Figure 13-19, Typical CRR Curve

For a specific earthquake-induced CSR value, the value located on the CRR boundary establishes a threshold in-situ testing value whereas in-situ testing results greater than the threshold value will not be susceptible to soil SSL and values less than the threshold value are subject to soil SSL.

Several empirical procedures have been developed to determine the cyclic resistance ratio (CRR) of Holocene (< 10,000 years) Sand-Like soils based on in-situ testing. In-situ testing that is acceptable to be used on SCDOT projects are Standard Penetration Testing (SPT) and Cone Penetrometer Testing (CPT). A comparison of advantages and disadvantages of these in-situ tests for determination of CRR are presented in Table 13-2. SPT and CPT measured results must be adjusted in accordance with Section 13.11.1.

Table 13-2, CRR Determination Based on Types of In-situ Testing (Modified after Youd and Idriss, 1997)

Feature	Type of In-situ Testing	
	SPT	CPT
Number of test measurements at liquefaction sites	Substantial	Many
Type of stress-strain behavior influencing test	Partially Drained, Large strain	Drained, Large Strain
Quality control and repeatability	Poor to Good	Very Good
Detection of variability of soil deposits	Good	Very Good
Soil types in which test is recommended	Non-Gravel	Non-Gravel
Test provides sample of soil	Yes	No
Test measures index or engineering property	Index	Index

The normalized cyclic resistance ratio curves ($CRR^* = CRR_{M=7.5,1\,tsf}$) for Sand-Like soils presented in Sections 13.11.2 and 13.11.3 are magnitude weighted (M_W=7.5) and normalized to a reference effective overburden stress of $\sigma'_V = 1$ tsf. These correlations were derived based on the relative state parameter index (ξ_R) by Idriss and Boulanger (2004). The corresponding CRR-ξ_R relationships derived from these two liquefaction correlations are shown in Figure 13-20 to illustrate the consistency between the SPT and CPT methods to predict field cyclic resistance ratio.

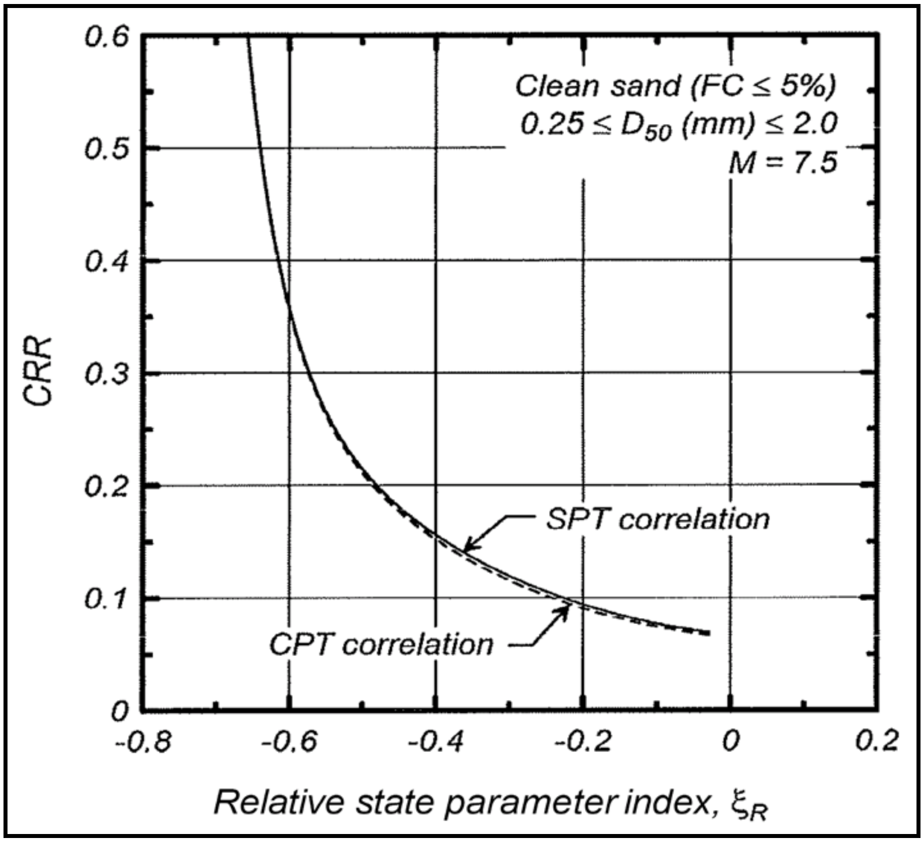


Figure 13-20, Field CRR-ξ_R Correlations Based on SPT and CPT (Idriss and Boulanger, 2004)

The normalized cyclic resistance ratio ($CRR^* = CRR_{M=7.5}$) for Clay-Like soil presented in Section 13.11.4 is magnitude weighted (M_W = 7.5).

Shear wave velocities (V_s) and the Becker Penetration Tests (BPT) methods for determination of the soil's resistance for liquefaction shall not be used for routine SCDOT soil SSL evaluations unless approved by the PCS/GDS.

The normalized cyclic resistance ratio (CRR^*) correlations must be further corrected to account for the effects of high overburden stress on Sand-Like soils (K_σ), effects of soil aging in Sand-Like soils (K_{DR}), and effects of initial static shear stress on Sand-Like Soils and Clay-Like Soils (K_α). The corrected and normalized cyclic resistance ratio curves (CRR^*_{eq}) are computed as indicated in the following general equation:

$$\boldsymbol{CRR^*_{eq}} = \boldsymbol{CRR^*}(\boldsymbol{K_\sigma})(\boldsymbol{K_{DR}})(\boldsymbol{K_\alpha}) \qquad \text{Equation 13-22}$$

Where,

CRR* = $CRR_{M=7.5,1\ tsf}$ = normalized cyclic resistance ratio magnitude weighted (M_W=7.5) and normalized to a reference effective overburden stress of σ'_V = 1 tsf. (Sand-Like Soil: Sections 13.11.2 & 13.11.3; Clay-Like Soil: Section 13.11.4)

K_σ = High overburden stress correction factor for Sand-Like Soils (Section 13.11.5)

K_{DR} = Age correction factor for Sand-Like Soils (Section 13.11.6)

K_α = Static shear stress ratio correction factor for Sand-Like and Clay-Like soils (Section 13.11.7)

13.11.1 In-Situ Testing Corrections For Evaluating Soil SSL

13.11.1.1 Correlations For Relative Density From SPT and CPT

Correlations to compute relative density (D_r) from SPT and CPT testing may be required for soil SSL analyses. The correlations proposed by Boulanger (2003b) to relate SPT N-values ($N^*_{1,60}$) and CPT tip resistance ($q_{c,1,N}$) to relative density (D_r) are provided below.

$$\boldsymbol{D_R} = \left(\frac{\boldsymbol{N}^*_{1,60}}{46}\right)^{0.5} \qquad \text{Equation 13-23}$$

$$\boldsymbol{D_R} = 0.478\left(\boldsymbol{q}_{\boldsymbol{c},1,\boldsymbol{N}}\right)^{0.264} - 1.063 \qquad \text{Equation 13-24}$$

Where,

$N^*_{1,60}$ = Corrected SPT N-value (Section 13.11.1.3) in blows per foot

$q_{c,1,N}$ = Normalized CPT tip resistance (Section 13.11.1.4) (unitless)

The relative density correlations (Equations 13-23 and 13-24) for SPT and CPT results can be combined to develop an SPT equivalent correlation for normalized CPT tip resistance as indicated by the following Equation.

$$\boldsymbol{N}^*_{1,60} = 46\left[0.478\left(\boldsymbol{q}_{\boldsymbol{c},1,\boldsymbol{N}}\right)^{0.264} - 1.063\right]^2 \qquad \text{Equation 13-25}$$

13.11.1.2 Overburden Reference Correction Factors For Sand-Like Soils For SPT And CPT

In order to obtain consistencies between SPT and CPT, these corrections shall only apply to Chapter 13. Normalized overburden reference (σ'_V = 1 tsf) correction factors developed by Boulanger (2003b) should be used when evaluating soil SSL as indicated by the following equation:

$$\boldsymbol{C_N} = \left(\frac{\boldsymbol{P_a}}{\boldsymbol{\sigma'_{vo}}}\right)^{\boldsymbol{m}} \qquad \text{Equation 13-26}$$

Where,

P_a = Atmospheric Pressure (1 atm = 1 tsf)

σ'_{vo} = Effective overburden stress (Use same units as used for P_a)

m = Exponent that was developed for sands at different relative densities (D_R) from calibration chamber data for SPT and CPT in-situ testing. Boulanger (2003b) recommended that exponent m to be computed as indicated below:

$$\boldsymbol{m} = 0.784 - (0.521 \cdot \boldsymbol{D_R}) \qquad \text{Equation 13-27}$$

Relative density (D_R) correlations for SPT and CPT in Section 13.11.1.1 can be used to compute exponent m proposed by Boulanger (2003b) based on Equation 13-27.

The normalized effective overburden factor (C_N) can be determined for SPT values of $N^*_{1,60} \leq$ 46 blows/foot by using the following equation.

$$\boldsymbol{C_N} = \left(\frac{\boldsymbol{P_a}}{\boldsymbol{\sigma'_{vo}}}\right)^{0.784 - 0.0768\sqrt{\boldsymbol{N^*_{1,60}}}} \leq 1.7 \qquad \text{Equation 13-28}$$

The normalized effective overburden factor (C_N) can be determined for normalized CPT tip resistance values of $21 \leq q_{c,1,N} \leq 254$ (unitless) by using the following equation.

$$\boldsymbol{C_N} = \left(\frac{\boldsymbol{P_a}}{\boldsymbol{\sigma'_{vc}}}\right)^{1.338 - 0.249(\boldsymbol{q_{c,1,N}})^{0.264}} \leq 1.7 \qquad \text{Equation 13-29}$$

Where,

$N^*_{1,60}$ = Corrected SPT N-value computed (Section 13.11.1.3) in blows per foot

$q_{c,1,N}$ = Normalized CPT tip resistance ($q_{c,1}$) (Section 13.11.1.4) (unitless)

The computation of the normalized effective overburden factor (C_N) indicated in Equations 13-28 and 13-29 for SPT and CPT, respectively, requires the iteration until there is convergence because C_N for SPT depends on $N^*_{1,60}$ and C_N for CPT depends on $q_{c,1,N}$. The normalized effective overburden factor (C_N) for SPT and CPT results are plotted in Figure 13-21.

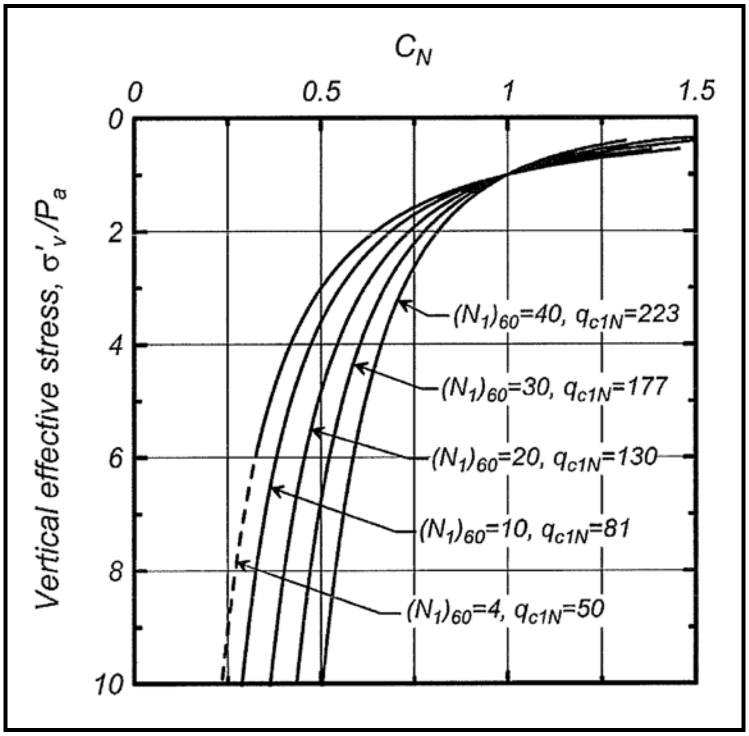


Figure 13-21, Overburden Correction Factor C_N
(Boulanger, 2003b with permission from ASCE)

13.11.1.3 SPT Blow Count Corrections for Soil SSL Evaluations

The measured Standard Penetration Test blow count, N_{meas}, must be standardized and adjusted for overburden prior to being used in soil SSL triggering analyses. Standardized SPT driving resistance, N^*_{60}, accounts for the effects of using non-standard SPT testing by applying corrections to account for hammer energy, SPT equipment, and procedural effects. The standardized SPT driving resistance, N^*_{60}, is adjusted to an effective reference overburden of σ'_V = 1 tsf. The corrected SPT blow count, $N^*_{1,60}$, is computed as indicated by the following equation.

$$\boldsymbol{N^*_{1,60} = N_{meas}\left(C_R C_S C_B C_E C_N\right)} \qquad \text{Equation 13-30}$$

Where,

N_{meas}	=	Measured SPT driving resistance. Units of blows per foot of penetration
C_R	=	Rod length correction factor
C_S	=	Nonstandard sampler correction factor
C_B	=	Borehole diameter correction factor
C_E	=	Hammer energy correction factor
C_N	=	Normalized effective overburden reference of σ'_V = 1 tsf correction factor

The correction factors C_R, C_S, C_B, and C_E are provided in Chapter 7. The normalized effective overburden reference (σ'_V = 1 tsf) correction factor, C_N, is provided in Section 13.11.1.2.

13.11.1.4 CPT Corrections for Soil SSL Evaluations

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 adjusting the measured resistances to a reference effective overburden stress of σ'_V = 1 tsf by using correction factor C_N developed for CPT in Section 13.11.1.2. The CPT tip resistance (q_c) measurements obtained in thin stiff layers is corrected by a thin layer correction factor (C_{thin}) as indicated in Chapter 7. The corrected CPT tip resistance ($q_{c,1}$) is computed by adjusting the measured CPT tip resistance (q_c) to a reference effective overburden stress of σ'_V = 1 tsf and correcting for thin layers as indicated by the following equation.

$$q_{c,1} = C_N C_{thin} q_c \qquad \text{Equation 13-31}$$

The normalized corrected CPT tip resistance ($q_{c,1,N}$) is computed by dividing the corrected CPT resistance ($q_{c,1}$) by the atmospheric pressure (P_a = 1 atm = 1 tsf) to eliminate units as indicated by the following equation.

$$q_{c,1,N} = \frac{q_{c,1}}{P_a} \qquad \text{Equation 13-32}$$

The corrected CPT sleeve resistance (f_s) is adjusted to a reference effective overburden stress of σ'_V = 1 tsf as indicated in the following equation.

$$f_{s,1} = C_N f_s \qquad \text{Equation 13-33}$$

The C_N correction factor used for tip resistance (q_c) and sleeve resistance (f_s) are the same value.

13.11.2 Sand-Like Soil - SPT Based CRR* Curves

The Cyclic Resistance Ratio (CRR) correlations for SPT in-situ testing presented by Idriss and Boulanger (2008) shall be used to evaluate Sand-Like soils. Deterministic cyclic resistance ratio curves ($CRR^* = CRR_{Mw=7.5,1\ tsf}$) are moment magnitude weighted, adjusted to a reference effective overburden stress of σ'_V = 1 tsf, and adjusted for fines content. Similarly to the cyclic stress ratio, CSR, a reference earthquake of moment magnitude, M_w, of 7.5 is used. The corrected SPT blow count ($N^*_{1,60}$) is adjusted to an equivalent clean sand (CS) blow count based on the fines content (FC) as indicated by the following equation.

$$N^*_{1,60,CS} = N^*_{1,60} + \Delta N^*_{1,60} \qquad \text{Equation 13-34}$$

Where,

$N^*_{1,60}$ = SPT blow count normalized to a reference effective overburden stress of σ'_V = 1 tsf, corrected for energy (60% - see Section 13.11.1.3). Units of blows/foot

$\Delta N^*_{1,60}$ = Fines content correction for 5% < FC < 35%. The variation in $\Delta N^*_{1,60}$ with fines content is shown in Figure 13-22.

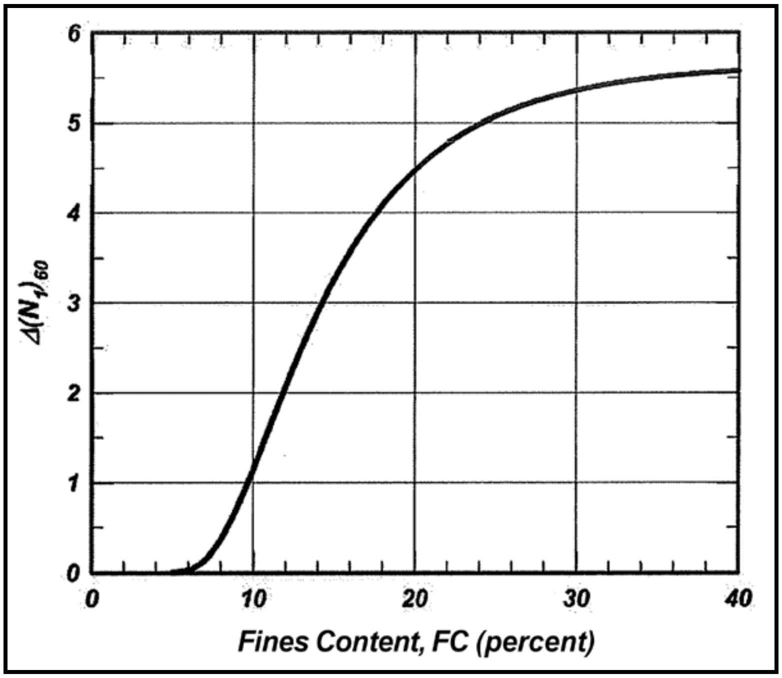


Figure 13-22, Variation in $\Delta N^*_{1,60}$ With Fines Content (Idriss and Boulanger, 2008)

In lieu of using Figure 13-22 the following equation may be used.

$$\Delta \boldsymbol{N}^*_{1,60} = \exp\left(1.63 + \left(\frac{9.7}{\boldsymbol{FC} + 0.01}\right) - \left(\frac{15.7}{\boldsymbol{FC} + 0.01}\right)^2\right) \leq 5.5 \qquad \text{Equation 13-35}$$

Where Fines content (FC) is in percent. Fines content (FC) is based on the soil fraction passing the No. 4 sieve.

Figure 13-23 shows the Idriss and Boulanger (2004) recommended deterministic CRR* curves for SPT in-situ testing based on an earthquake moment magnitude, $M_w = 7.5$, effective overburden reference stress, $\sigma'_v = 1.0$ tsf, and fines content FC < 5%.

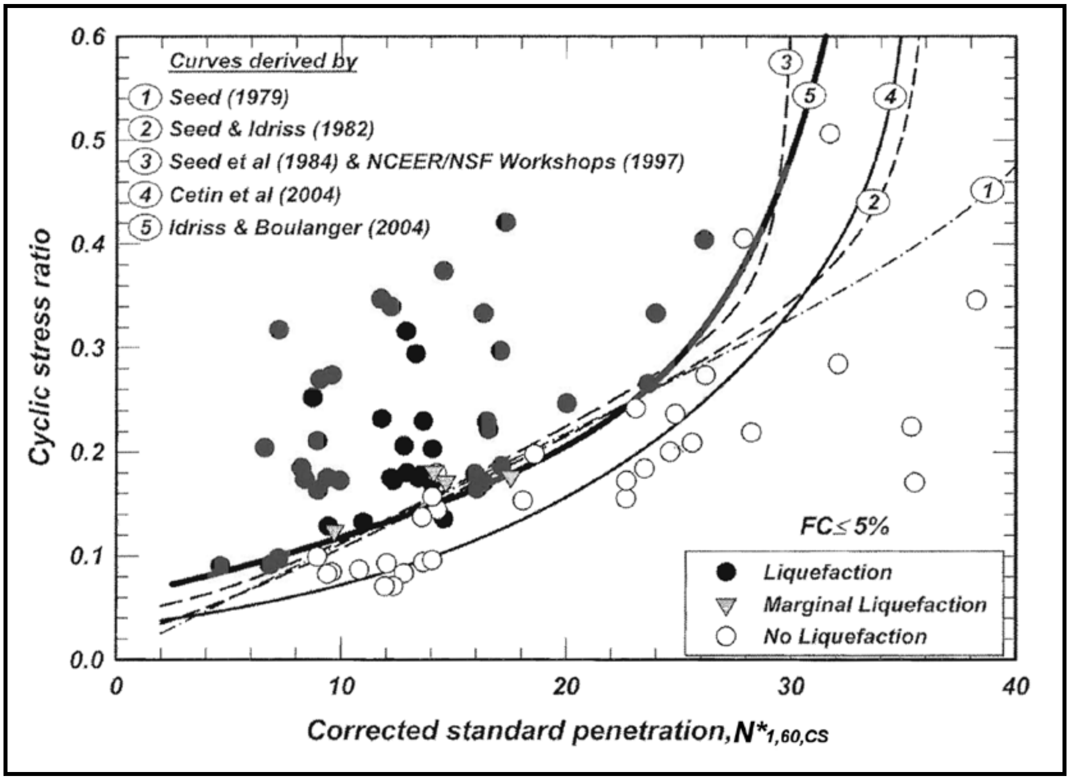


Figure 13-23, SPT Liquefaction Triggering Correlation (CRR*)

M_w = 7.5; σ'_{vo} = 1.0 tsf; FC ≤ 5%

(Idriss and Boulanger, 2004)

In lieu of using Figure 13-23 the following equation may be used.

$$CRR^* = CRR_{M_w=7.5,1tsf} = \exp\left[\left(\frac{N^*_{1,60,CS}}{14.1}\right)+\left(\frac{N^*_{1,60,CS}}{126}\right)^2-\left(\frac{N^*_{1,60,CS}}{23.6}\right)^3+\left(\frac{N^*_{1,60,CS}}{25.4}\right)^4-2.8\right] \quad \text{Equation 13-36}$$

13.11.3 Sand-Like Soil - CPT Based CRR* Curves

The cyclic resistance ratio (CRR) correlations for CPT in-situ testing presented by Idriss and Boulanger (2008) shall be used to evaluate Sand-Like soils. Deterministic cyclic resistance ratio curves ($CRR^* = CRR_{Mw=7.5,1\ tsf}$) are moment magnitude weighted, adjusted to a reference effective overburden stress of σ'_V = 1 tsf, and adjusted for fines content. Similarly to the cyclic stress ratio, CSR, a reference earthquake of moment magnitude, M_w, of 7.5 is used. The normalized corrected CPT tip resistance ($q_{c,1,N}$) is adjusted to an equivalent clean sand (CS) tip resistance based on the fines content (FC) as indicated by the following equation.

$$q_{c,1,N,CS} = q_{c,1,N} + \Delta q_{c,1,N} \quad \text{Equation 13-37}$$

Where,

$q_{c,1,N}$ = Normalized corrected CPT tip resistance (Section 13.11.1.4) (unitless)

$\Delta q_{c,1,N}$ = Fines content correction for FC > 5%

The variation in $\Delta q_{c,1,N}$ with fines content (FC) based on Idriss and Boulanger, 2008 can be obtained from Figure 13-24 for FC > 5%.

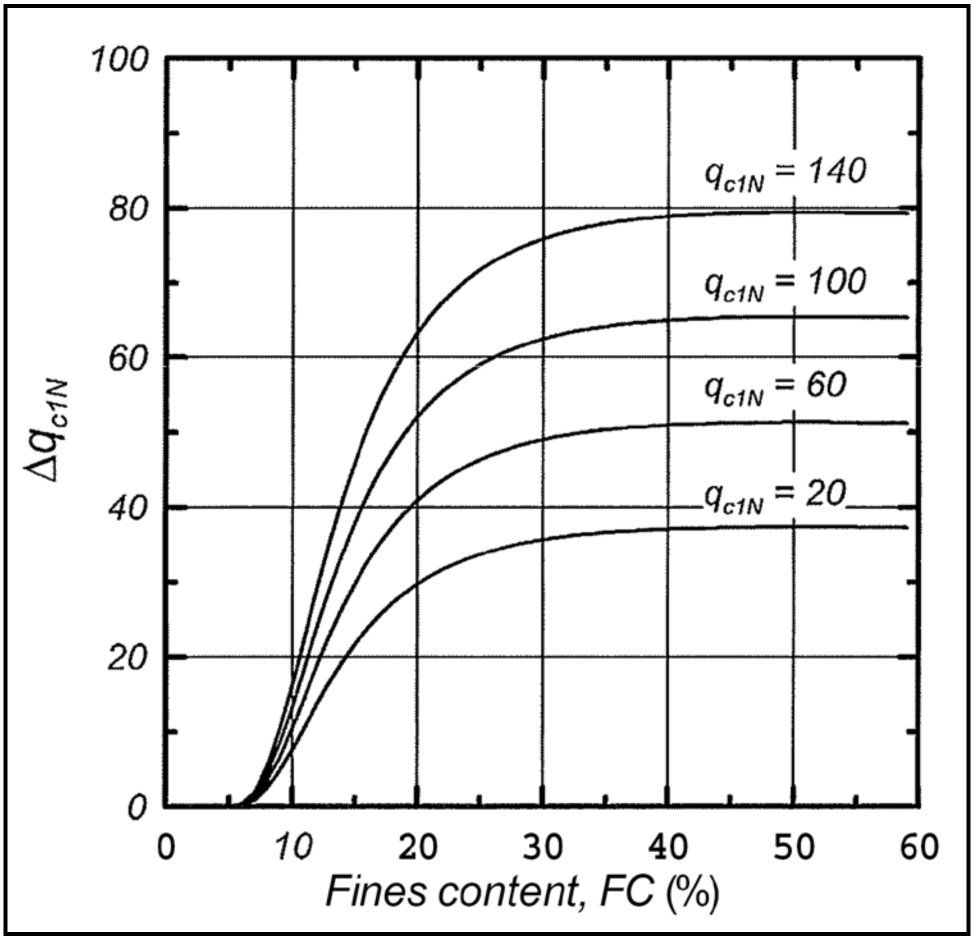


Figure 13-24, Variation in $\Delta q_{c,1,N}$ With Fines Content (Idriss and Boulanger, 2008)

In lieu of using Figure 13-24 the following equation may be used.

$$\Delta q_{c,1,N} = \left(5.4 + \left(\frac{q_{c,1,N}}{16}\right)\right) \cdot \exp\left(1.63 + \left(\frac{9.7}{FC + 0.01}\right) - \left(\frac{15.7}{FC + 0.01}\right)^2\right) \quad \text{Equation 13-38}$$

Where Fines content (FC) is in percent. Fines content (FC) is based on the soil fraction passing the No. 4 sieve.

Figure 13-25 shows the Idriss and Boulanger (2004) recommended deterministic CRR* curves for CPT in-situ testing based on an earthquake moment magnitude, $M_w = 7.5$, effective overburden reference stress, σ'_v=1.0 tsf, and fines content FC < 5%.

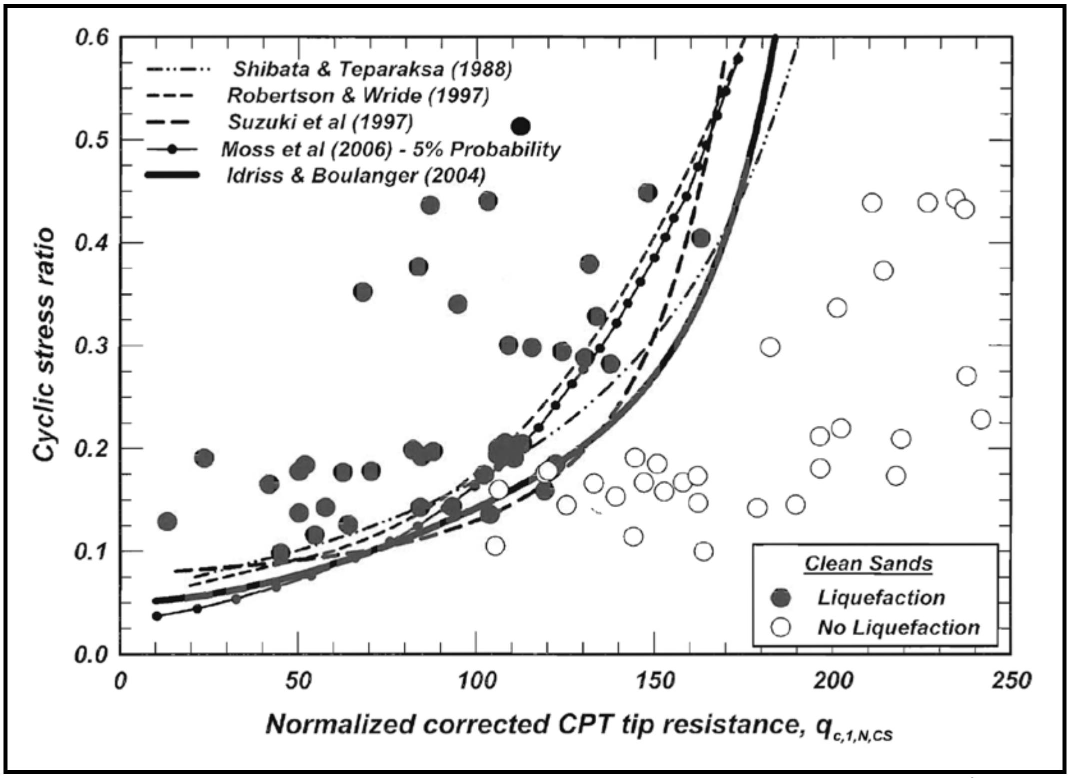


Figure 13-25, CPT Liquefaction Triggering Correlation (CRR*)
M_w = 7.5; σ'$_{vo}$ = 1.0 tsf; $FC \leq 5\%$
(Idriss and Boulanger, 2004)

In lieu of using Figure 13-25 the following equation may be used.

$$\boldsymbol{CRR}^{*} = \boldsymbol{CRR}_{M_w=7.5,\sigma'_{vo}} = \exp\left[\left(\frac{q_{c,1,N,CS}}{540}\right)+\left(\frac{q_{c,1,N,CS}}{67}\right)^2-\left(\frac{q_{c,1,N,CS}}{80}\right)^3+\left(\frac{q_{c,1,N,CS}}{114}\right)^4-3.0\right] \quad \text{Equation 13-39}$$

13.11.4 Clay-Like Soil CRR* Curves

The cyclic resistance ratio (CRR) correlations presented by Idriss and Boulanger, (2008) shall be used to evaluate Clay-Like soils. Deterministic cyclic resistance ratio curves (CRR* = $CRR_{M=7.5}$) are magnitude weighted. Similarly to the cyclic stress ratio, CSR, a reference earthquake moment magnitude, M_w, of 7.5 is used. The CRR of Clay-Like soils will typically be determined by using empirical correlations. CRR of Clay-Like soils can also be determined by cyclic laboratory testing with approval from the PCS/GDS. Boulanger and Idriss (2007) developed empirical correlations based on the undrained shear strength profile and the consolidation stress history profile.

The preferred empirical correlation for determining the cyclic resistance ratio curves (CRR* = $CRR_{Mw=7.5}$) for Clay-Like soils is based on the undrained shear strength profile using the relationship shown in Figure 13-26, where undrained shear strengths have been obtained from laboratory testing. If in-situ testing methods (SPT or CPT) are used to obtain undrained shear strengths then CRR* correlations using the consolidation stress history profile presented in Figure 13-26 should be used as a check on the in-situ testing shear strength correlations.

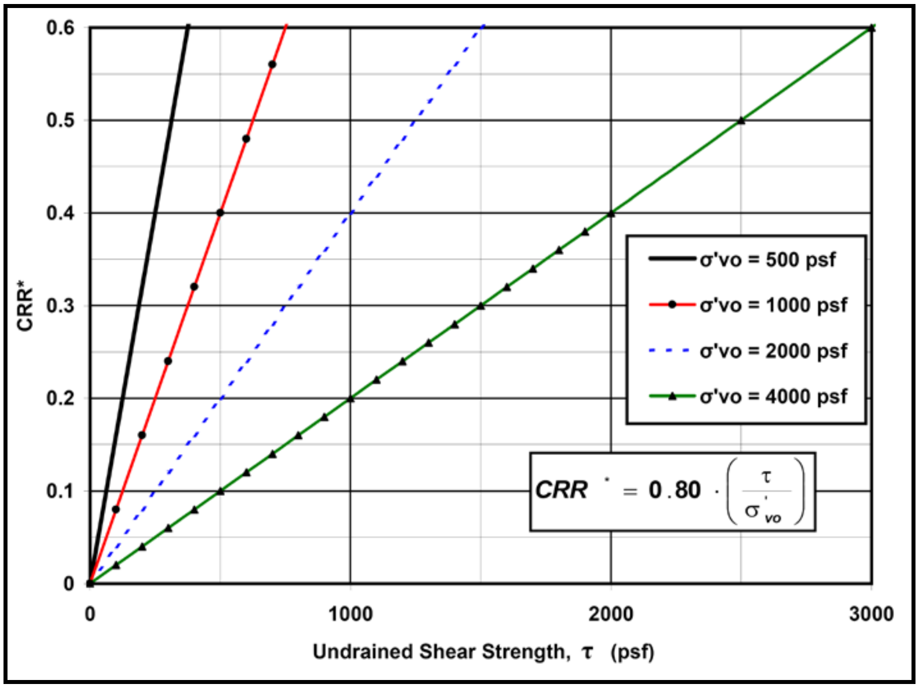


Figure 13-26, CRR* Clay-Like – Shear Strength Correlation (Modified from Boulanger and Idriss, 2007 with permission from ASCE)

In lieu of using Figure 13-26, the following equation may be used to determine the cyclic resistance ratio curves ($CRR^* = CRR_{Mw=7.5}$) for Clay-Like soils.

$$\boldsymbol{CRR}^* = \boldsymbol{CRR}_{M_w=7.5} = 0.80\left(\frac{\tau}{\sigma'_{vo}}\right) \qquad \text{Equation 13-40}$$

Where,

τ = Undrained shear strength (S_u). Laboratory and in-situ testing methods for determining S_u are presented in Chapter 7.

σ'_{vo} = effective overburden stress

Boulanger and Idriss (2007) have suggested using the empirical correlations developed from SHANSHEP laboratory testing (Ladd et al., 1977) shown in Figure 13-27. These correlations are based on a relationship between the undrained shear strength ratio and the consolidation stress history. The overconsolidation ratio (OCR) provides a measure of the consolidation stress history. These correlations require a consolidation stress history profile that is sometimes difficult to accurately evaluate without performing consolidation tests at various depths. It has also been observed that the undrained shear strength ratio can vary based on the type of clay formation used as shown Chapter 7 and in Figure 13-27. This method should only be used for preliminary analyses or to evaluate the cyclic resistance ratio (CRR^*) determined by the undrained shear strength ratio, particularly if in-situ testing is used to estimate the undrained shear strength of Clay-Like soils.

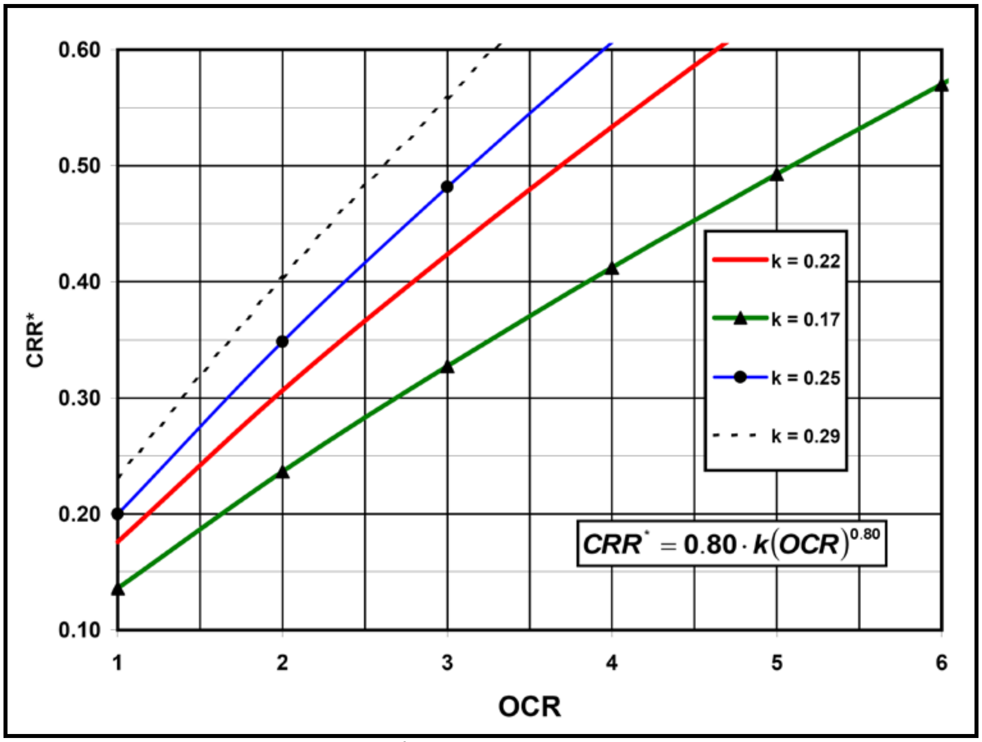


Figure 13-27, CRR* Clay-Like Soils – OCR Correlation

In lieu of using Figure 13-27, the following equation may be used to compute the cyclic resistance ratio curves ($CRR^* = CRR_{M=7.5,1\ tsf}$) for Clay-Like soils based on OCR.

$$\boldsymbol{CRR^{*} = 0.80 \cdot k(OCR)^{n}}$$ Equation 13-41

Where,

k = Shear strength ratio for normally consolidated soils (OCR=1) typically ranges between 0.17 and 0.29. Use k=0.22 (DSS testing) as recommended by Boulanger and Idriss (2007) unless laboratory testing and local correlations indicate otherwise.

OCR = Overconsolidation ratio (σ'_p / σ'_{vo}) (See Chapter 7)

n = soil constant typically taken as 0.80 for unstructured and uncemented soils.

13.11.5 High Overburden Correction For Sand-Like Soils (K_σ)

The high overburden correction, K_σ, accounts for the increased susceptibility of Sand-Like soils to cyclic liquefaction, at the same CSR, with large increases in effective overburden stress. For Clay-Like soils there is no increase, therefore, a high overburden correction, K_σ, = 1.0 shall be used.

The high overburden correction factors for Sand-Like soils presented by Idriss and Boulanger (2008) shall be used. These high overburden correction factors are based on the relative state parameter index (ξ_R). These correction factors were then correlated with corrected SPT blow counts ($N^*_{1,60}$ – Section 13.11.1.3) and normalized corrected CPT tip resistance ($q_{c,1,N}$ – Section 13.11.1.4). The high overburden corrections, K_σ, for effective overburden σ'_{vo} > 1 tsf, are plotted for selected values of $N^*_{1,60}$ and $q_{c,1,N}$ in Figure 13-28.

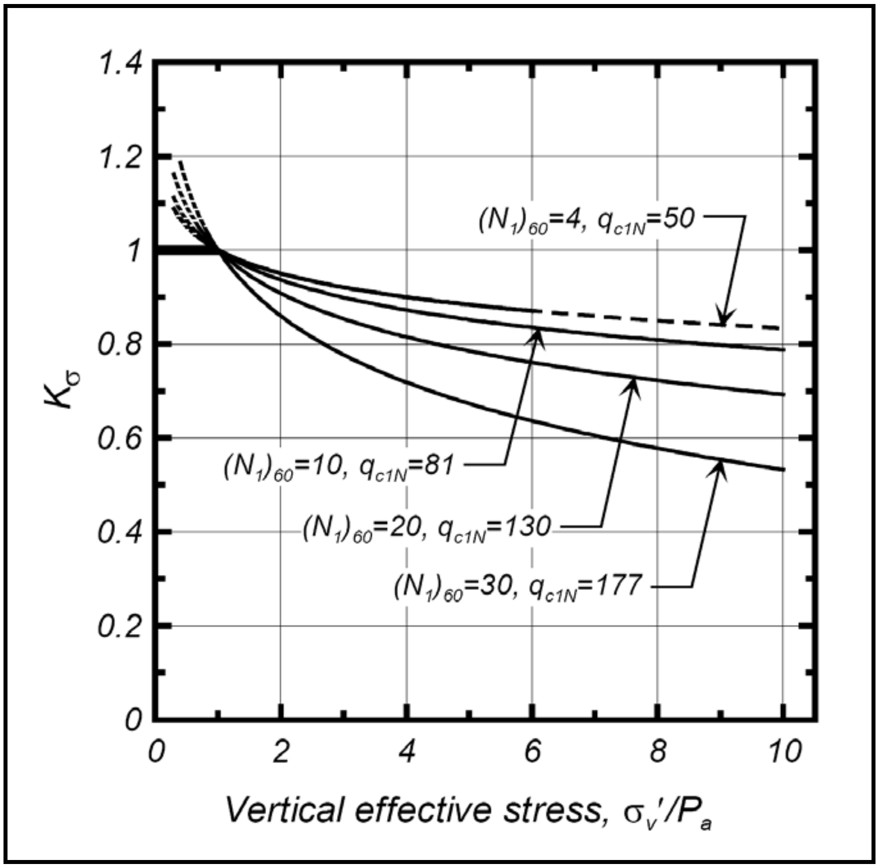


Figure 13-28, High Overburden Correction (K_σ) (σ'_{vo} > 1 tsf) (Boulanger, 2003a with permission from ASCE)

In lieu of using Figure 13-28, the following equation may be used to compute the K_σ of Sand-Like soils. These correlations are based on Q $\approx$ 10, $K_o \approx 0.45$, $D_R \leq 0.9$, and $(\sigma'_{vo}/P_a) \leq 10$.

$$\boldsymbol{K_\sigma} = \mathbf{1} - \boldsymbol{C_\sigma} \ln\left(\frac{\boldsymbol{\sigma'_{vo}}}{\boldsymbol{P_a}}\right) \leq \mathbf{1.1} \qquad \text{Equation 13-42}$$

Where,

- σ'_{vo} = Effective overburden stress (or σ'_v), units of tsf.
- P_a = Atmospheric pressure, taken as 1 tsf
- C_σ = Coefficient used to correlate D_R, $N^*_{1,60}$, and $q_{c,1,N}$ to K_σ
- D_R = Relative density, where $D_R \leq 0.90$ (90%)
- $N^*_{1,60}$ = Corrected SPT blow count, where $N^*_{1,60} \leq 37$ blows/foot
- $q_{c,1,N}$ = Corrected and normalized CPT tip resistance, where $q_{c,1,N} \leq 211$ unitless

The coefficient C_σ can be expressed in terms of relative density (D_R), corrected SPT blow count ($N^*_{1,60}$), and corrected and normalized CPT tip resistance ($q_{c,1,N}$) based on Boulanger and Idriss (2004a) as indicated by the following equations.

$$C_{\sigma} = \frac{1}{18.9 - 17.3 D_R} \leq 0.3 \qquad \text{Equation 13-43}$$

$$C_{\sigma} = \frac{1}{18.9 - 2.55\left(N^{*}_{1,60}\right)^{0.5}} \leq 0.3 \qquad \text{Equation 13-44}$$

$$C_{\sigma} = \frac{1}{37.3 - 8.27\left(q_{c,1,N}\right)^{0.264}} \leq 0.3 \qquad \text{Equation 13-45}$$

13.11.6 Age Correction Factor For Sand-Like Soils (K_{DR})

The susceptibility of Sand-Like soils to cyclic liquefaction has been found to be a function of geologic age and origin. Soils that were formed during the Quaternary period (past 1.6 million years ago - Mya), include the Holocene and Pleistocene epochs and shall be considered to have a moderate to very high potential for liquefaction. Pre-Pleistocene age (more than 1.6 Mya) deposits shall be considered to have a lower susceptibility to liquefaction. Youd and Perkins (1978) proposed geologic susceptibility chart for cyclic liquefaction of sedimentary cohesionless soil deposits that was based on soil deposition and geologic age as indicated in Table 13-3. The soil resistance to cyclic liquefaction tends to increase with increase in age as observed in Table 13-3.

Soil formations that are Pre-Pleistocene (>1.6 Mya) potentially will have a lower susceptibility to experience cyclic liquefaction unless the soils are found in areas where there is evidence of the soils having experienced cyclic liquefaction. Soils found in Pre-Pleistocene areas that have been subjected to cyclic liquefaction will have the same susceptibility to cyclic liquefaction as soils formed during the Holocene period. Figure 13-8 provides the location of paleoliquefaction sites that have been studied and Figure 13-9 provides a map developed by the USGS that identifies areas in South Carolina that potentially have experienced Quaternary liquefaction.

Simplified liquefaction-triggering methods used to compute the Cyclic Resistance Ratio (CRR) for Sand-Like soils such as those proposed by Youd and Idriss (1997) and Idriss and Boulanger (2008) were developed from case histories of relatively young Holocene (< 10,000 years ago) soils. A study by Leon et al. (2006) has demonstrated that Pleistocene Sand-Like soils in the upper 20 feet of several locations within the South Carolina Coastal Plain may have increased resistance to liquefaction due to aging. The location of paleoliquefaction sites in the Coastal Plain that were used by Leon et al. (2006) are shown in Figure 13-8.

Table 13-3, Liquefaction Susceptibility of Sedimentary Deposits
(Modified after Youd and Perkins, 1978 with permission from ASCE)

Type of Deposit [1]	General Distribution of Cohesionless Sediments in Deposits	Likelihood that Cohesionless Sediments, When Saturated, Will be Susceptible to Liquefaction (By Age of Deposit)			
		Modern < 500 yr	**Holocene 500 yr to 10 ka**	**Pleistocene 10ka – 1.6 Mya**	**Pre-Pleistocene > 1.6 Mya**
(a) Continental Deposits					
River Channel	Locally Variable	Very High	High	Low	Very Low
Floodplain	Locally Variable	High	Moderate	Low	Very Low
Alluvial Fan & Plain	Widespread	Moderate	Low	Low	Very Low
Marine Terraces & Plains	Widespread	---	Low	Very Low	Very Low
Delta and Fan-delta	Widespread	High	Moderate	Low	Very Low
Lacustrine and Playa	Variable	High	Moderate	Low	Very Low
Colluvium	Variable	High	Moderate	Low	Very Low
Talus	Widespread	Low	Low	Very Low	Very Low
Dunes	Widespread	High	Moderate	Low	Very Low
Loess	Variable	High	High	High	Unknown
Glacial Till	Variable	Low	Low	Very Low	Very Low
Tuff	Rare	Low	Low	Very Low	Very Low
Tephra	Widespread	High	High	Unknown	Unknown
Residual Soils	Rare	Low	Low	Very Low	Very Low
Sebka	Locally Variable	High	Moderate	Low	Very Low
(b) Coastal Zone					
Delta	Widespread	Very High	High	Low	Very Low
Estuarine	Locally Variable	High	Moderate	Low	Very Low
Beach - High Wave-energy	Widespread	Moderate	Low	Very Low	Very Low
Beach - Low Wave-energy	Widespread	High	Moderate	Low	Very Low
Lagoonal	Locally Variable	High	Moderate	Low	Very Low
Fore Shore	Locally Variable	High	Moderate	Low	Very Low
(c) Artificial					
Uncompacted Fill	Variable	Very High	---	---	---
Compacted Fill	Variable	Low	---	---	---

Definitions:
ka = thousands of years ago
Mya = millions of years ago

[1] Notes:
The above types of soil deposits may or may not exist in South Carolina. All of the soil deposits included by the original authors have been kept for completeness.

A study was recently conducted at the Savannah River Site (SRS) by Lewis et al. (2007) to re-evaluate the soil aging effects on the liquefaction resistance of Sand-Like soils that were encountered within shallow subsurface Tertiary soils from the Eocene (53 Mya) and Miocene (23 Mya) epochs. The results of these studies indicate that there is a significant increase in the CRR of sand with time as indicated by Figure 13-29.

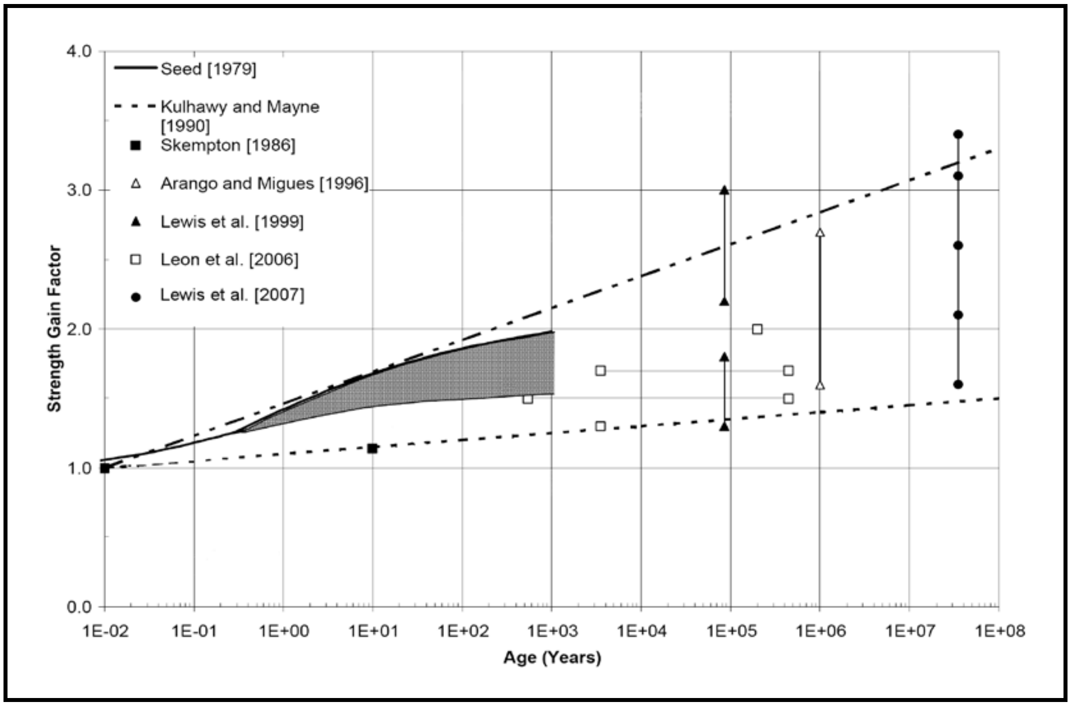


Figure 13-29, Sand-Like Soil Strength Gain With Age (Adapted from Lewis et al., 2007)

Hayati and Andrus (2008) reviewed the results of nine published studies on the effects of aging on liquefaction resistance of soils and developed a regression line (Solid Line) shown in Figure 13-30 that represents the average variation in liquefaction age correction factor (K_{DR}) with time (t). The age correction factor (K_{DR}) is the ratio of resistance-corrected cyclic resistance ratio of the aged soil (CRR_{DR}) to the cyclic resistance ratio of recently deposited soil ($CRR_{Holocene}$) as indicated by the following equation.

$$K_{DR} = \frac{CRR_{DR}}{CRR_{Holocene}}$$ Equation 13-46

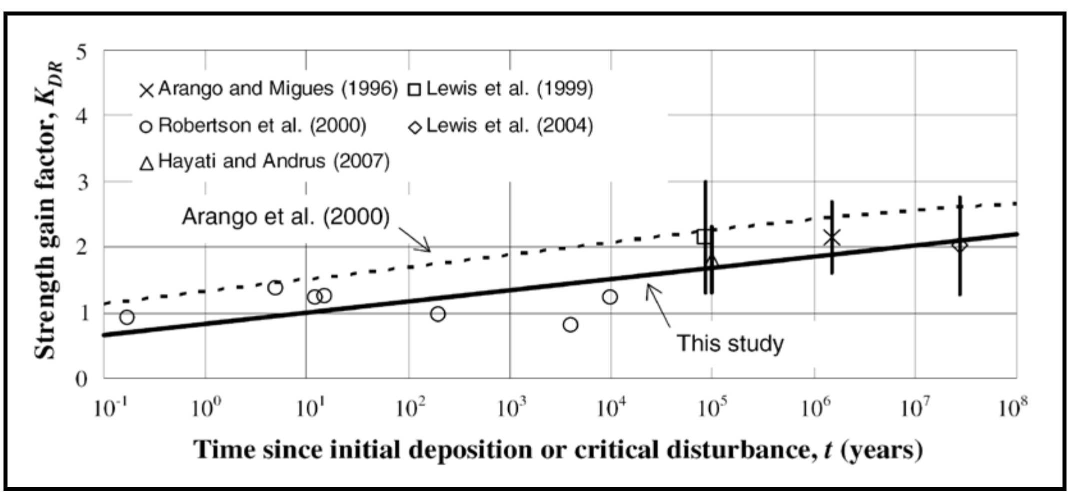


Figure 13-30, Relationship Between Strength Gain Factor and Time (Hayati and Andrus, 2008 with permission from ASCE)

A liquefaction age correction factor of K_{DR} = 1.0 corresponds to a soil deposit with an age of approximately 10 years. The time, t, is the time since initial deposition (K_{DR} = 1.0) or critical disturbance in years. Critical disturbance occurs when the effects of soil aging are removed as a result of grain-to-grain contacts being broken and reformed such as has been observed when Sand-Like soils experience cyclic liquefaction.

The age correction factor (K_{DR}) shown in Figure 13-30 can be computed using the following equation.

$$\boldsymbol{K_{DR}} = \mathbf{0.17}\log_{\mathbf{10}}(\boldsymbol{t}) + \mathbf{0.83}$$ Equation 13-47

Because it is difficult to evaluate the age of the soil, Andrus et al. (2009) have developed a concept for evaluating aged sands, MERV, ratio of measured shear wave velocity, $V_{s\text{-}meas}$, to estimated shear wave velocity from SPT or CPT ($V_{S\text{-}SPT}$ or $V_{S\text{-}CPT}$) as indicated by the following equations.

$$\boldsymbol{MERV} = \frac{\boldsymbol{V_{S-Meas}}}{\boldsymbol{V_{S-SPT}}}$$ Equation 13-48

$$\boldsymbol{MERV} = \frac{\boldsymbol{V_{S-Meas}}}{\boldsymbol{V_{S-CPT}}}$$ Equation 13-49

Where,

$V_{s\text{-}meas}$ = Measured shear wave velocity by Crosshole (CH), Seismic CPT (SCPT), Suspension Logger (SL), or other approved method.

$V_{S\text{-}SPT}$ = Estimated shear wave velocity using SPT ($N^*_{1,60,CS}$) by using the following equation:

$$\boldsymbol{V_{S-SPT}} = \boldsymbol{V_{s,1cs-SPT}} = \mathbf{87.8}\left(\boldsymbol{N^*_{1,60,CS}}\right)^{\mathbf{0.253}}$$ Equation 13-50

$V_{S\text{-}CPT}$ = Estimated shear wave velocity using CPT ($N^*_{1,60,CS}$) by using the following equation:

$$\boldsymbol{V_{S-CPT}} = \boldsymbol{V_{s,1cs-CPT}} = \mathbf{62.6}\left(\boldsymbol{q_{c,1N,CS}}\right)^{\mathbf{0.231}}$$ Equation 13-51

Andrus et al. (2009) proposed the following relationship between MERV and time (t in years) since initial deposition or critical disturbance.

$$\boldsymbol{MERV} = \left(\mathbf{0.0820}\cdot\log_{\mathbf{10}}(\boldsymbol{t})\right) + \mathbf{0.935}$$ Equation 13-52

Andrus et al. (2009) proposed a relationship between the age correction factor, K_{DR}, (Equation 13-47) and MERV (Equation 13-52) as indicated by the following equation.

$$\boldsymbol{K_{DR}} = \left(\mathbf{2.07}\cdot\boldsymbol{MERV}\right) - \mathbf{1.11}$$ Equation 13-53

An approximate relationship between South Carolina Coastal Plain Geology, MERV, and K_{DR} is presented in Table 13-4. The use of an age correction factor, $K_{DR} > 1.0$ for Sand-Like soils will require that the MERV for the soil formation be computed and compared with values presented in Table 13-4. Sand-Like soil formations with a MERV ≤ 1.3 shall be assumed to be either Holocene age soils or aged soil formations that have been subjected to critical disturbance such as cyclic liquefaction. Values listed in Table 13-4 should not be exceeded without the approval of the PCS/GDS.

Table 13-4, Coastal Plain Sand-Like Soil Age Correction Factor, K_{DR} (MERV)
(adapted from South Carolina Geological Survey – Willoughby et al. , 1999)

Time Scale		Time	SC Coastal Plain Physiographic Province				
Period	**Epoch**	**(MYA)**	**Upper**	**Middle**	**Lower**	**MERV**	**K_{DR}**
Quaternary	Holocene	0.01	Coastal Sands River Bottoms Alluvium	Coastal Sands River Bottoms Alluvium	Coastal Sands River Bottoms Alluvium Tidal Marsh Estuarine Deposits Fresh Water Stream Swamp Deposits	1.02	1.00
	Pleistocene	1.6		Socastee Formation Wando Formation Penahloway Formation	Silver Bluff Beds Princess Anne Formation Pamlico Formation Socatee Formation Wando Formation Ten Mile Hill Formation Canepatch Formation Ladson Formation Penhaloway Formation Wicomico Formation Waccamaw Formation Daniel Island Beds	1.31	1.60
Tertiary	Pliocene	5.3	Pinehurst Formation	Okefenokee Terrace Duplin Formation Bear Bluff Formation	Lower Beds at Windy Hill Lower Waccamaw Formation Pringletown Beds Raysor Formation Goose Creek Limestone Bear Bluff Formation Givhans Beds Duplin Formation Wabasso Beds	1.45	1.90
	Miocene	23	Altamaha Formation Coharie Formation Citronelle Formation	Coharie Formation	Coosawhatchie Formation Rudd Branch Beds Markhead Formation Parachula Formation Edisto Formation Hawtorne Formaiton	1.50	2.00
	Oligocene	36.6		Suwanee Limestone	Chandler Bridge Formation Ashley Formation Suwanee Limestone Lazaretto Creek Formation	1.55	2.10
	Eocene	53	Tobaco Road Sand Dry Branch Formation Orangeburg District Bed McBean Formation Santee Limestone Warley Hill Formation Huber Formation Congaree Formation Fourmile Branch Formation Tinker Formation Hazelhurst Formation	Ocala Limestone Orangeburg District Bed Santee Limestone Warley Hill Formation Congaree Formation Hazelhurst Formation	Parkers Ferry Formation Harleyville Formation Cross Formation Santee Limestone Fishburne Formation Drayton Limestone Hazelhurst Formation Ocala Limestone		
	Paleocene	65	Snapp Formation Lang Syne Formation Sawdust Landing Fm. Black Mingo Formation Elenton Formation	Lang Syne Formation Williamsburg Formation Sawdust Landing Fm. Black Mingo Formation	Williamsburg Formation Rhems Formation Black Mingo Formation	1.57	2.15
Mesozoic	Upper Cretaceous	99.6	Tar Heel Formation Black Creek Formation Middendorf Formation	Pee Dee Formation Donoho Creek Formation Black Creek Formation Middendorf Formation Bladen Formation Tar Heel Formation	Pee Dee Formation Steel Creek Formation Black Creek Formation Middendorf Formation Donoho Creek Formation Tar Heel Formation Bladen Formation Tuscaloosa Formation	1.59	2.18
	Lower Cretaceous Jurassic Triassic	250.0	Transitional Material between Coastal Plain Sediment and Piedmont Rocks			-	-
Paleozoic	Permian Pennsylvanian Mississippian Devonian Silurian Ordovicin Cambrian	570.0	Piedmont			-	-

13.11.7 Static Shear Stress Ratio Correction Factor (K_α)

The static shear stress ratio correction factor, K_α, accounts for the effects of initial static shear stresses on cyclic resistance of the soils beneath sloping ground. The static shear stresses are typically expressed as the static shear stress ratio (α) that is defined as the initial static shear stress (τ_s) divided by the effective normal consolidation stress (σ'_{vc}) as indicated by the following equation.

$$\alpha = \frac{\tau_S}{\sigma'_{vc}} = \frac{\tau_S}{\sigma'_{vo}} \qquad \text{Equation 13-54}$$

The initial static shear stress (τ_s) is the static soil shear stress that exists prior to the earthquake shaking occurring and can be computed as indicated in Section 13.11.7.1. The effective normal consolidation stress is typically assumed to be equal to the effective overburden stress (σ'_{vc} = σ'_{vo}) because most design situations assume enough time has elapsed that the soils have been consolidated under the sustained loading. For under consolidated soils, the existing effective consolidation stress (σ'_{vc}) shall be used.

The static shear stress ratio correction factor, K_α, is defined as the ratio of CRR at some value of α divided by the CRR at a value of α = 0 as indicated by the following equation.

$$K_\alpha = \frac{(CRR)_\alpha}{(CRR)_{\alpha=0}} \qquad \text{Equation 13-55}$$

The static shear stress ratio correction factor, K_α, for Sand-Like soils and Clay-Like soils can be computed in accordance with Sections 13.11.7.2 and 13.11.7.3, respectively.

13.11.7.1 Initial Static Shear Stress (τ_s) of Soils Susceptible to Soil SSL

The initial static shear stress (τ_s) for each soil layer susceptible to soil SSL (Sand-Like soils and Clay-Like soils) can be computed by performing a slope stability analysis of the pre-failure geometry. The slope stability analysis should be performed in accordance with Chapter 17 with Spencer's method required. The slope stability analysis should be evaluated using both circular and sliding wedge potential failure surfaces. The slope stability analysis should then be performed by using undrained shear strengths for cohesive soil layers and fully mobilized drained shear strengths for cohesionless soils. Determine the slope stability ratio, $(D/C)_{Stability}$ using the following equation.

$$\left(\frac{D}{C}\right)_{Stability} = \frac{1}{FS} = \varphi_{Stability} \qquad \text{Equation 13-56}$$

The initial static shear stress (τ_s) is defined as the soil shear stress along the failure surface that corresponds to slope stability ratio of $(D/C)_{Stability}$ = 1. The initial static shear stress (τ_s) along the critical failure surface can be computed by reducing soil shear strengths based on the computed slope stability ratio, $(D/C)_{Stability}$ for the pre-failure geometry. The reduced undrained shear strengths for cohesive soil layers, c_s, can be computed using the following equation.

$$c_s = c \cdot \left(\frac{D}{C}\right)_{Stability} = \frac{c}{FS} = c \cdot \varphi_{Stability} \qquad \text{Equation 13-57}$$

Where,

c	=	Undrained shear strengths, S_u = c
$\left(\frac{D}{C}\right)_{Stability}$	=	Slope stability ratio for the pre-failure geometry
FS	=	Factor of Safety for the pre-failure geometry
$\varphi_{Stability}$	=	Resistance Factor for the pre-failure geometry

The initial static shear stress (τ_s) along the critical failure surface for cohesive soils is computed by the following equation.

$$\tau_s = c_s \qquad \text{Equation 13-58}$$

The reduced drained shear strength for a cohesionless soil layer is computed by reducing the internal friction angle, ϕ_s, and can be computed using the following equation.

$$\phi_s = \arctan\left[\tan\phi \cdot \left(\frac{D}{C}\right)_{Stability}\right] = \arctan\left[\frac{\tan\phi}{FS}\right] = \arctan\left[\tan\phi \cdot \left(\varphi_{Stability}\right)\right] \qquad \text{Equation 13-59}$$

Where,

ϕ	=	Internal friction angle
$\left(\frac{D}{C}\right)_{Stability}$	=	Slope stability ratio for the pre-failure geometry
FS	=	Factor of Safety for the pre-failure geometry
$\varphi_{Stability}$	=	Resistance Factor for the pre-failure geometry

The initial static shear stress (τ_s) along the critical failure surface for cohesionless soils is computed by the following equation.

$$\tau_s = \sigma'_{vo} \cdot \tan\phi_s \qquad \text{Equation 13-60}$$

Alternatively, some slope stability software allows the input of the shear strength ratio directly (τ/σ'_{vo}). The static shear strength ratio (α) for cohesive soils can be computed by the following equation.

$$\alpha = \left(\frac{\tau_s}{\sigma'_{vo}}\right) = \left(\frac{\tau}{\sigma'_{vo}}\right) \cdot \left(\frac{D}{C}\right)_{Stability} = \frac{\tau}{\sigma'_{vo} \cdot FS} = \varphi_{Stability} \cdot \left(\frac{\tau}{\sigma'_{vo}}\right) \qquad \text{Equation 13-61}$$

The static shear strength ratio (α) for cohesionless soils can be computed by the following equation.

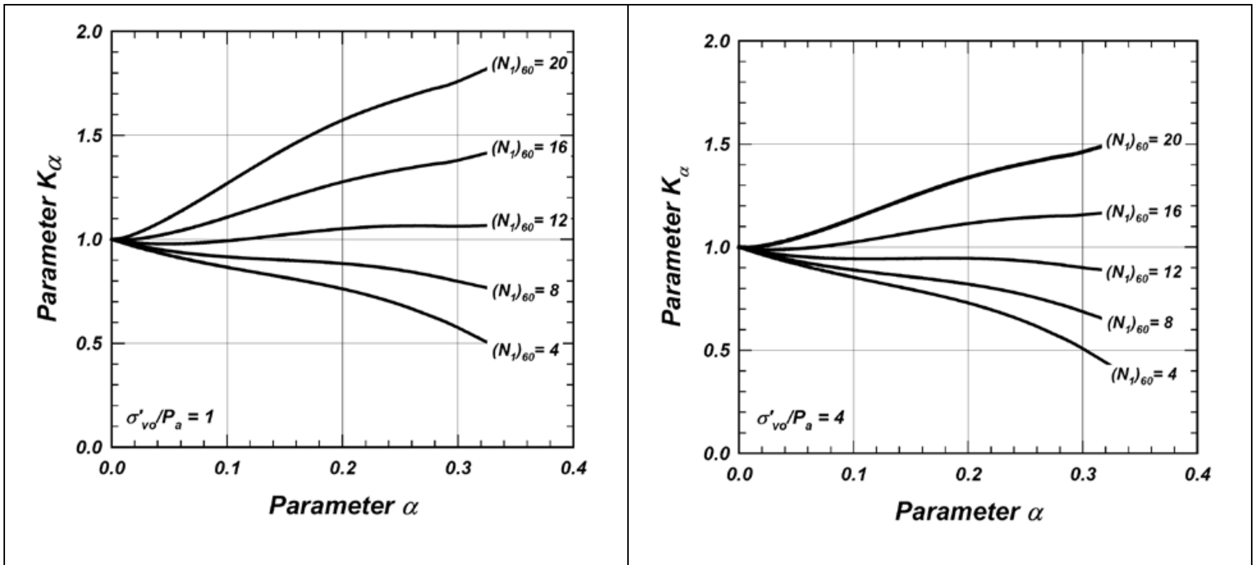
$$\alpha = \left(\frac{\tau_S}{\sigma'_{vo}}\right) = (\tan\phi)\cdot\left(\frac{D}{C}\right)_{Stability} = \frac{\tan\phi}{FS} = \varphi_{Stability} \cdot \tan\phi \qquad \text{Equation 13-62}$$

The initial static soil shear stress computed should be checked by using the reduced soil shear strengths (τ_S or α) to perform a slope stability analysis and determine if the slope stability ratio, $(D/C)_{Stability}$, for the critical failure surface corresponds to a slope stability ratio of $(D/C)_{Stability} = 1$. If the slope stability ratio of $(D/C)_{Stability} \neq 1$, the soil shear strength should be further adjusted until a slope stability ratio of $(D/C)_{Stability} = 1$ is achieved.

13.11.7.2 K_α For Sand-Like Soils

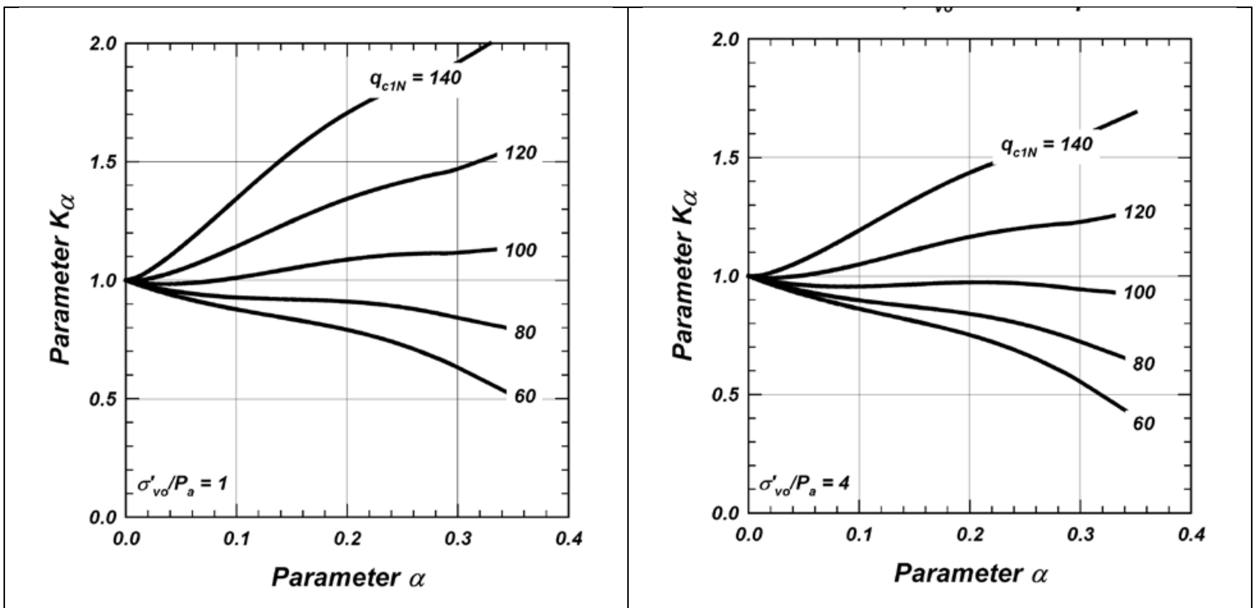
Harder and Boulanger (1997) observed variations in cyclic shear stresses as a function relative density (D_R) and effective overburden stress (σ'_{vo}) when they summarized available cyclic laboratory test data. It was observed that cyclic resistance of dense sands can increase significantly as the static shear stress ratio (α) increases and that cyclic resistance of loose sands decreases as the static shear stress ratio (α) decreases.

Boulanger (2003b) developed static shear stress ratio correction factor, K_α, for Sand-Like soils based on the relative state parameter index (ξ_R). These correction factors were then correlated for use with normalized SPT N-values ($N^*_{1,60}$ – Section 13.11.1.3) normalized and effective overburden corrected CPT tip resistance ($q_{c,1,N}$ – Section 13.11.1.4). The static shear stress ratio correction factor, K_α, for selected effective overburden stresses of σ'_{vo} = 1 tsf and σ'_{vo} = 4 tsf, and Q=10 (Sand), are provided for SPT values of $N^*_{1,60}$ in Figure 13-31 and CPT values of $q_{c,1,N}$ in Figure 13-32.



(a) Effective Vertical Stress, σ'_{vo} = 1 tsf	(b) Effective Vertical Stress, σ'_{vo} = 4 tsf

Figure 13-31, Variations of K_α with SPT Blow Count ($N^*_{1,60}$)
(Boulanger, 2003b)



(a) Effective Vertical Stress, σ'_{vo} = 1 tsf	(b) Effective Vertical Stress, σ'_{vo} = 4 tsf

Figure 13-32, Variations of K_α with CPT Tip Resistance ($q_{c,1,N}$)
(Boulanger, 2003b)

In lieu of using Figures 13-31 and 13-32, the following equation may be used to compute the K_α. The following equations were developed from data that limits the static shear stress ratio to $\alpha \leq 0.35$ and relative state parameter index to $-0.6 \leq \xi_R \leq 0.1$.

$$K_\alpha = a + b \cdot \left[\exp\left(\frac{-\xi_R}{c} \right) \right] \qquad \text{Equation 13-63}$$

Where,

a = $1267 + 636\alpha^2 - (634 \cdot \exp(\alpha)) - (632 \cdot \exp(-\alpha))$

b = $\exp(-1.11 + 12.3\alpha^2 + (1.31 \cdot \ln(\alpha + 0.0001)))$

c = $0.138 + 0.126\alpha + 2.52\alpha^3$

α = Static shear stress ratio as per Equation 13-62 and Section 13.11.7.1 (limited to $\alpha \leq 0.35$)

ξ_R = Relative state parameter index (ξ_R) used to correlate D_R, $N^*_{1,60}$, and $q_{c,1,N}$ to K_α (Limited to: $-0.6 \leq \xi_R \leq 0.1$)

$$\xi_R = \frac{1}{Q - \ln\left(\frac{100(1 + 2K_o)\sigma'_{vo}}{3P_a} \right)} - D_R \qquad \text{Equation 13-64}$$

$$\xi_R = \frac{1}{Q - \ln\left(\frac{100(1 + 2K_o)\sigma'_{vo}}{3P_a} \right)} - \left(\frac{N^*_{1,60}}{46} \right)^{0.5} \qquad \text{Equation 13-65}$$

$$\xi_R = \frac{1}{Q - \ln\left(\frac{100(1 + 2K_o)\sigma'_{vo}}{3P_a} \right)} - \left(0.478 (q_{c,1,N})^{0.264} - 1.063 \right) \qquad \text{Equation 13-66}$$

Where,

D_R = Relative density, where $D_R \leq 0.90$ (90%)

$N^*_{1,60}$ = Standardized and normalized SPT blow count, where $N^*_{1,60} \leq 37$ blows/foot

$q_{c,1,N}$ = Normalized CPT tip resistance, where $q_{c,1,N} \leq 211$ (unitless)

Q = Empirical Constant: Q=10 for Quartz and feldspar (Sand), Q=8 for limestone, Q=7 for anthracite, and Q=5.5 for chalk.

K_o = At-rest lateral earth pressure coefficient

σ'_{vo} = Effective vertical overburden stress

13.11.7.3 K_α For Clay-Like Soils

Boulanger and Idriss (2007) developed static shear stress ratio correction factor, K_α, for Clay-Like soils based on laboratory testing of Drammen clay (Goulois et al., 1985) that was consolidated under a sustained static shear stress. The relationship developed from these laboratory tests for K_α versus $(\tau_S/S_U)_{\alpha=0}$ for Clay-Like soils are shown in Figure 13-33.

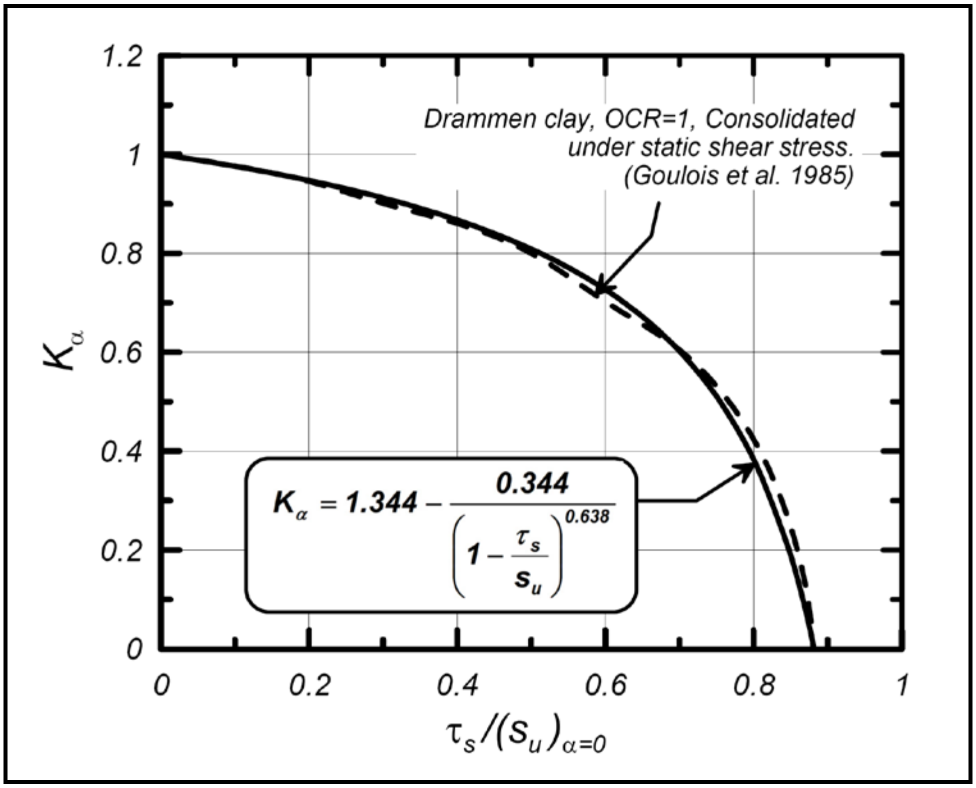


Figure 13-33, K_α versus $(\tau_S/S_U)_{\alpha=0}$ For Clay-Like Soil (NC Drammen Clay) (Boulanger, 2007)

The static shear stress ratio correction factor, K_α, relationship shown in Figures 13-33 is presented in the following equation as a function of (τ_s/S_U) and ($\alpha = \tau_s/\sigma'_{vc}$).

$$\boldsymbol{K}_\alpha = 1.344 + \frac{0.344}{\left(1 - \frac{\tau_S}{\boldsymbol{S}_U}\right)^{0.638}} = \frac{0.344}{\left(1 - \left(\frac{\alpha}{\boldsymbol{S}_U / \sigma'_{vc}}\right)\right)^{0.638}} \qquad \text{Equation 13-67}$$

Where,

τ_s	=	Initial static shear stress
S_u	=	Undrained shear strength
σ'_{vc}	=	Effective vertical consolidating stress
S_u/σ'_{vc}	=	Undrained shear strength ratio (See Chapter 7)
α	=	Static shear stress ratio as per Equation 13-61and Section 13.11.7.1 (limited to $\alpha \leq 0.35$)

Boulanger (2003b) recommended using an empirical shear strength ratio relationship (S_u/σ'_{vc}) such as those developed by Ladd and Foot (1974) that take the form of the following equation:

$$\left(\frac{S_u}{\sigma'_{vc}}\right) = k(OCR)^n \qquad \text{Equation 13-68}$$

Where,

k	=	Shear strength ratio for normally consolidated soils (OCR=1). Typically range between 0.17 and 0.29. Use k=0.22 (DSS testing) as recommended by Boulanger (2007) unless laboratory testing available.
OCR	=	Overconsolidation ratio (σ'_p / σ'_{vo}) (See Chapter 7)
n	=	soil constant typically taken as 0.80 for unstructured and uncemented soils.

The static shear stress ratio correction factor, K_α, presented in Equation 13-67 can be combined with the empirical shear strength ratio shown in Equation 13-68 to develop static stress correction factor (K_α) that is a function of the consolidation stress history as shown in Figure 13-34 and Equation 13-69.

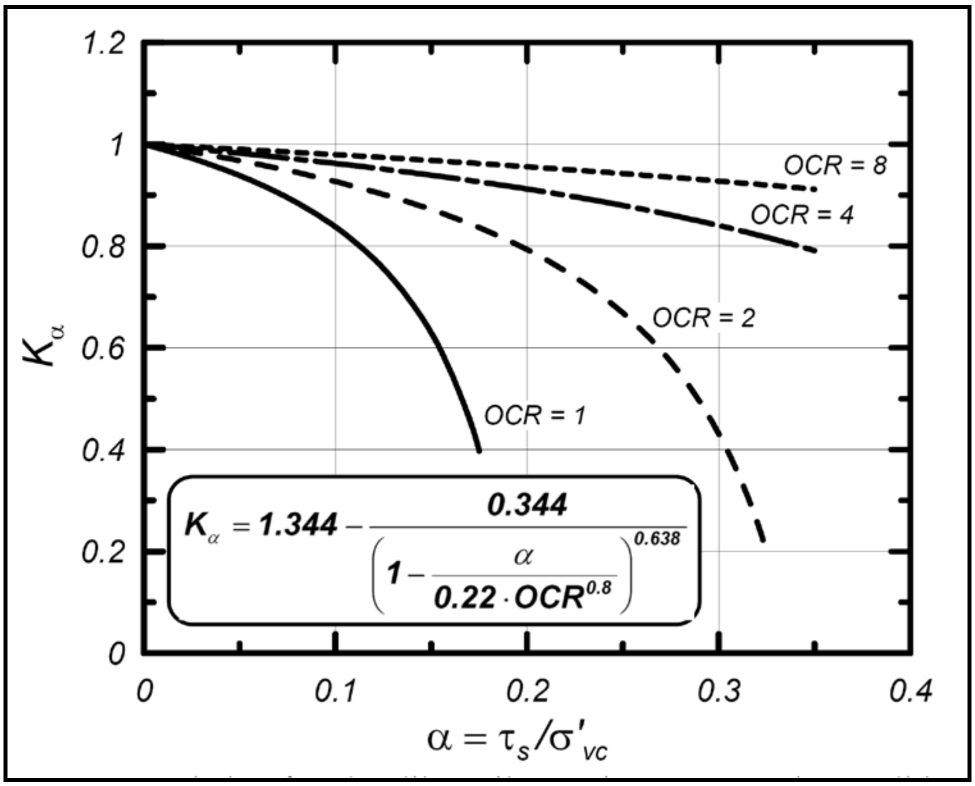


Figure 13-34, K_α versus $(\tau_S/S_U)_{\alpha=0}$ For Clay-Like Soil ($1 \le OCR \le 8$) (Boulanger, 2007)

$$K_\alpha = 1.344 + \frac{0.344}{\left(1-\left(\frac{\alpha}{k(OCR)^n}\right)\right)^{0.638}} = 1.344 + \frac{0.344}{\left(1-\left(\frac{\alpha}{0.22(OCR)^{0.8}}\right)\right)^{0.638}} \qquad \text{Equation 13-69}$$

13.12 SOIL SHEAR STRENGTH FOR SEISMIC ANALYSES

When performing seismic analyses, for soils that are not subject to losses in shear strength, the appropriate undrained shear (τ) or drained shear (τ') strengths should be used, in accordance with Chapter 7. Soils that are subject to cyclic strain-softening should use residual soil shear strengths. Undrained/drained soil shear strengths can be evaluated in accordance with the field and laboratory testing procedures specified in Chapter 5.

During strong earthquake shaking, cyclic liquefaction of Sand-Like soils or cyclic softening of Clay-Like soils may result in a sudden loss of strength and stiffness. Laboratory testing to determine residual shear strength of soils that have been subject to cyclic liquefaction or cyclic softening is difficult and not typically performed. The standard-of-practice is to use correlated residual undrained shear strengths of cohesionless soils as indicated in Sections 13.12.1 and 13.12.2 and to use correlated cyclic shear strength of cohesive soils as indicated in Sections 13.12.3 and 13.12.4. Guidance in selection of soil shear strengths for seismic analyses is presented in Section 13.12.5.

13.12.1 <u>Sand-Like Soil Cyclic Shear Strength Triggering</u>

The shear strength of Sand-Like soils that should be used in seismic analyses is dependent on the results of the liquefaction triggering and on pore pressure generation. The liquefaction triggering resistance ratio $(D/C)_{SL}$ for level site conditions and steeply sloped site conditions are presented in Sections 13.7 and 13.9, respectively. Figure 13-31 shows a relationship between liquefaction triggering resistance ratio $(D/C)_{SL}$ and the excess pore pressure ratio, R_u, that was proposed by Marcuson et al. (1990) for level sites.

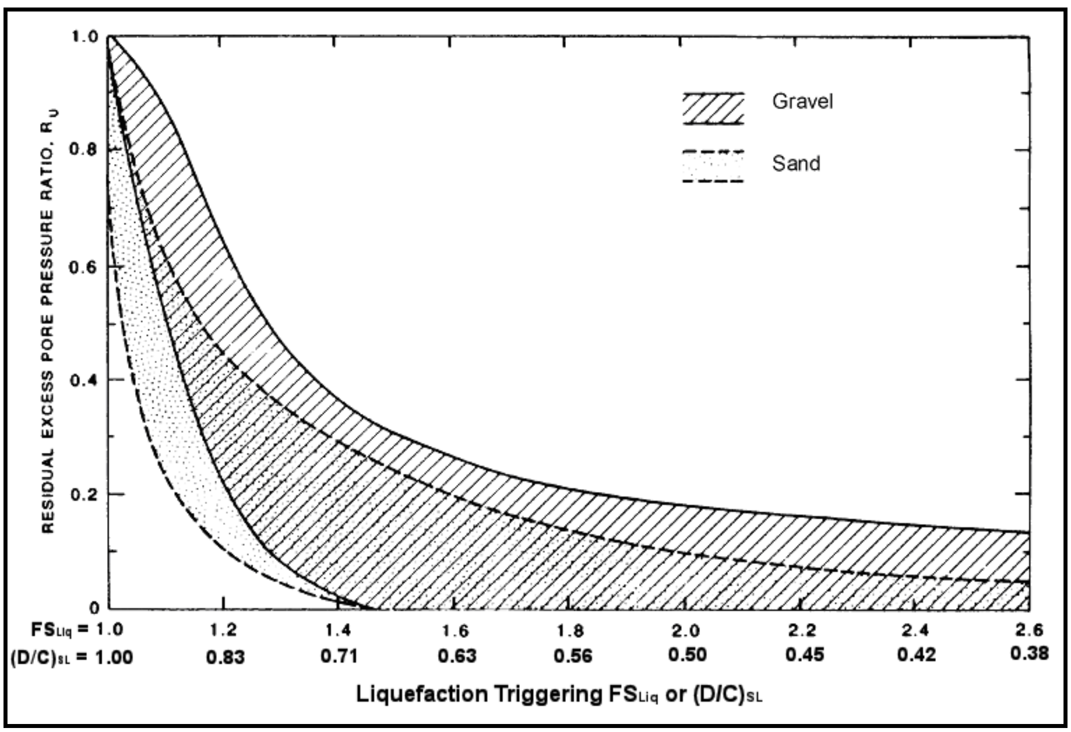


Figure 13-35, Excess Pore Pressure Ratio - Liquefaction Triggering (Modified Marcuson et al., 1990)

The excess pore pressure ratio, $R_u = \Delta u/\sigma'_{vo}$ is the ratio of excess pore water pressure (Δu) divided by the effective overburden stress (σ'_{vo}). The liquefaction triggering resistance ratio $(D/C)_{SL}$ of Sand-Like soils can be defined based on the excess pore pressure ratio generated as either Full Liquefaction, Limited Liquefaction, or No Liquefaction. The Limited Liquefaction potential represents a transition zone between the triggering resistance factor for the onset of liquefaction ($\varphi_{SL\text{-}Sand}$) and the no soil SSL resistance factor, $\varphi_{NLS\text{-}Sand}$ for Sand-Like Soils as indicated in Figure 13-32. The resistance factors (φ) indicated in Figure 13-32 are for illustration purposes. Resistance factors used for design shall be those presented in Chapter 9.

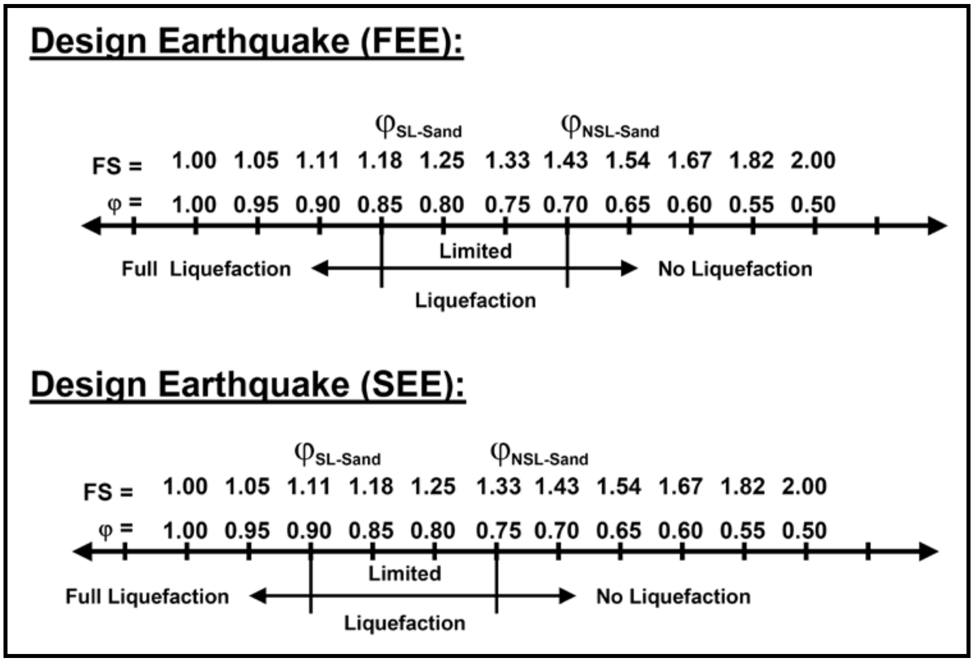


Figure 13-36, Shear Strength of Sand-Like Soils

The Limited Liquefaction case represents partial pore pressure generation and the shear strength for this case can be computed based on the following equation.

$$\tau_{rl-\lim} = \sigma'_{vo}(1 - R_u)\tan(\phi_{rl}) \qquad \text{Equation 13-70}$$

Where,

R_u = Excess pore pressure ratio

ϕ_{rl} = Internal friction angle for cyclic liquefaction

The equivalent limited liquefaction internal friction angle $\phi'_{rl\text{-}lim}$ can be approximated by reducing the internal friction angle (ϕ_{rl}) as a function of the excess pore water pressure ratio using the following equation.

$$\phi'_{rl-\lim} = \tan^{-1}\left[(1 - R_u)\left(\frac{\tau_{rl}}{\sigma'_{vo}}\right)\right] = \tan^{-1}\left[(1 - R_u)\tan(\phi_{rl})\right] \qquad \text{Equation 13-71}$$

Guidelines for determining the shear strength of Sand-Like soils are provided in Table 13-5.

Table 13-5, Sand-Like Shear Strengths

Liquefaction Potential	Liquefaction Triggering Criteria	Soil Shear Strength
Full Liquefaction	$(D/C)_{SL\text{-}Sand} < \varphi_{SL\text{-}Sand}$ $(R_u \approx 1.0)$	Use Idriss and Boulanger (2008) residual shear strength of liquefied soils (τ_{rl}) correlations. Section 13.12.2.3 Equation 13-72 $\phi_{rl} = \tan^{-1}\left(\frac{\tau_{rl}}{\sigma'_{vo}}\right)$
Limited Liquefaction	$\varphi_{SL\text{-}Sand} \le (D/C)_{SL\text{-}Sand} < \varphi_{NSL\text{-}Sand}$ $(0.20 \le R_u < 1.0)$	Use $\phi_{rl\text{-}lim}$ based on partial pore pressure generation using a $R_u = 0.4$ Equation 13-73 $\phi'_{rl-\lim} = \tan^{-1}\left[(1-R_u)\left(\frac{\tau_{rl}}{\sigma'_{vo}}\right)\right] = \tan^{-1}\left[(1-R_u)\tan(\phi_{rl})\right]$
No Liquefaction	$(D/C)_{SL\text{-}Sand} \le \varphi_{NSL\text{-}Sand}$ $(R_u < 0.20)$	Drained shear strength (τ') or residual shear strength (τ'_r) based on strain level. See Chapter 7.

13.12.2 Sand-Like Soil Cyclic Liquefaction Shear Strength

The following three methods are currently used to estimate the residual shear strength of liquefied Sand-Like soils.

1. SPT - Seed and Harder (1990)
2. SPT and CPT - Olson and Stark (2002, 2003)
3. SPT and CPT – Idriss and Boulanger (2008)

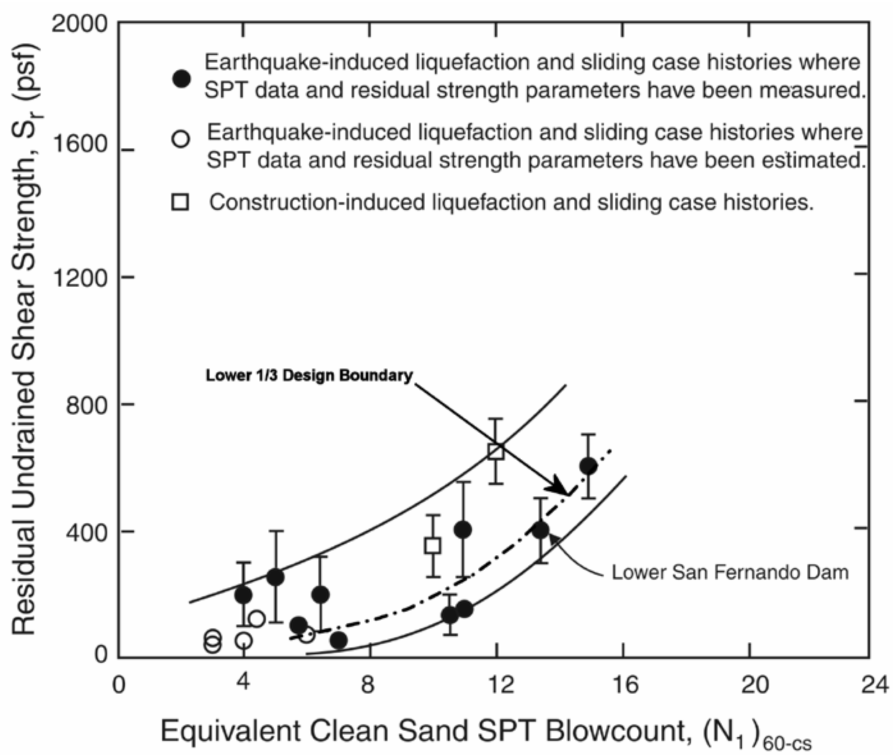
The Idriss and Boulanger (2008) method is the preferred method because it incorporates case histories from Seed (1987), Seed and Harder (1990), and Olson and Stark (2002). The Idriss and Boulanger (2008) method is also more advanced in that it uses residual shear strength ratios and allows residual shear strengths to be evaluated for void redistribution effects. All three methods are presented below to provide the designer with the appropriate background to evaluate the appropriate residual shear strength for liquefied Sand-Like soils.

13.12.2.1 Seed and Harder (1990) – Liquefied Residual Shear Strength

The Seed and Harder (1990) chart has been one of the most widely used methods to estimate the upper and lower bound relationship between $N^*_{1,60,cs}$ and residual shear strength of liquefied soils (τ_{rl}). The Seed and Harder (1990) correlation is shown in Figure 13-33. Typically the lower one-third (1/3) design boundary shown in Figure 13-33 has been used to compute the residual shear strength of liquefied soils.

The Seed and Harder (1990) correlation uses a corrected fines content SPT blow count, $N^*_{1,60,cs}$, computed as shown in Equation 13-67. Standardized and normalized SPT blow counts, $N^*_{1,60}$, should be computed in accordance with Chapter 7. Values for N_{Corr} can be found in Figure 13-33.

$$N^*_{1,60,CS} = N^*_{1,60} + N_{Corr}$$ Equation 13-74



Recommended Fines Correction for S_R
Evaluation Using SPT Data

Percent Fines	N_{Corr} (blows/ft)
10%	1
25%	2
50%	4
75%	5

Figure 13-37, Residual Shear Strength ($\tau_{rl} = S_{rl}$) vs. Corrected Clean Sand SPT (Modified Seed and Harder, 1990)

13.12.2.2 Olson and Stark (2002, 2003) – Liquefied Residual Shear Strength

The Olson and Stark (2002, 2003) methods allow for the computation of the liquefaction shear strength ratio (τ_{rl}/σ'_{vo}), which is the ratio of liquefied shear strength (τ_{rl}) to effective overburden stress (σ'_{vo}). The relationship between liquefaction shear strength ratio and the normalized SPT blow count ($N^*_{1,60}$) is provided in Figure 13-34. The average trend line for Figure 13-34 can be computed using the following equation.

$$\left(\frac{\tau_{rl}}{\sigma'_{vo}}\right) = 0.03 + 0.0075\left(N^*_{1,60}\right) \pm 0.03 \qquad \text{Equation 13-75}$$

Where,

$N^*_{1,60}$ = Normalized SPT blow count and values of $N^*_{1,60} \leq 12$ blows per foot.

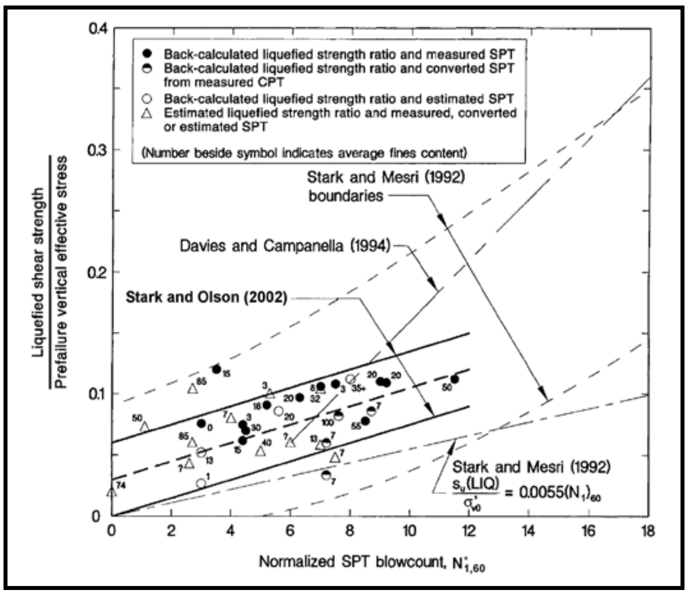


Figure 13-38, Liquefied Shear Strength Ratio - SPT Blow Count (Olson and Stark, 2002, 2003 with permission from ASCE)

The relationship between liquefaction shear strength ratio and the normalized CPT tip resistance ($q_{c,1}$) is provided Figure 13-35. The average trend line for Figure 13-35 can be computed using the following equation.

$$\left(\frac{\tau_{rl}}{\sigma'_{vo}}\right) = \mathbf{0.03} + \mathbf{0.0143}\left(q_{c,1}\right) \pm \mathbf{0.03} \qquad \text{Equation 13-76}$$

Where,

$q_{c,1}$ = Normalized CPT tip resistance (in units of MPa) for values $q_{c,1} \leq 6.5$ MPa (approximately 68 tsf) .

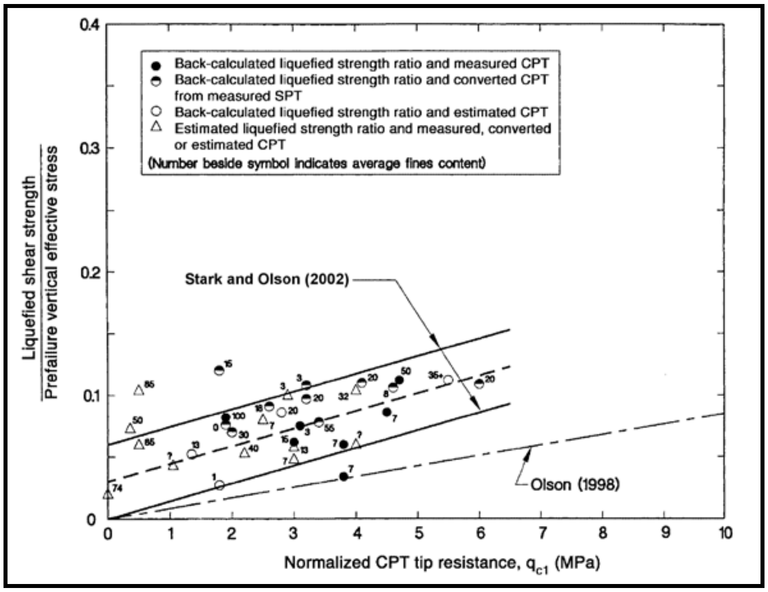


Figure 13-39, Liquefied Shear Strength Ratio - CPT Tip Resistance (Olson and Stark, 2002, 2003 with permission from ASCE)

13.12.2.3 Idriss and Boulanger (2008) – Liquefied Residual Shear Strength

The Idriss and Boulanger (2008) method allows for the computation of the liquefaction residual shear strength ratio (τ_{rl} / σ'_{vo}), which is the ratio of liquefied residual shear strength (τ_{rl} = S_{rl}) to effective overburden stress (σ'_{vo}). The liquefaction residual shear strength ratio (τ_{rl} / σ'_{vo}) has been correlated to SPT and CPT in-situ testing.

The Idriss and Boulanger (2008) relationship for liquefaction residual shear strength ratio (τ_{rl} / σ'_{vo}) for SPT and CPT in-situ testing includes the following:

Case 1: Condition in which the effects of void redistribution can be confidently judged to be negligible. This condition occurs at sites where the site stratigraphy would not impede dissipation of excess pore water pressure and the dissipation of excess pore water pressure would be accompanied by densification of the soils.

Case 2: Condition in which the effects of void redistribution can be significant. This condition occurs at sites where the site stratigraphy would impede dissipation of excess pore water pressure. Sites that meet this condition include sites with relatively thick layers of liquefiable soils that are overlain by lower permeability soils that would impede the dissipation of excess pore water pressure by trapping the upwardly seeping pore water beneath the lower permeability soil. This condition would lead to localized loosening, strength loss, and possibly even the formation of water film beneath the lower permeability soil.

The SPT correlation uses a corrected fines content SPT blow count, $N^*_{1,60,cs}$, computed as shown in Equation 13-70. Corrected and normalized SPT blow counts, $N^*_{1,60}$, should be computed in accordance with Section 13.11.1.3. Values for $\Delta N_{1,60\text{-}rl}$ can be found in Table 13-6.

$$N^{*}_{1,60,CS} = N^{*}_{1,60} + \Delta N_{1,60-rl}$$ Equation 13-77

Table 13-6, Values of $\Delta N_{1,60-rl}$
(Seed, 1987)

Fines Content, FC (% passing No. 200 sieve)	$\Delta N_{1,60-rl}$
10	1
25	2
50	4
75	5

The liquefaction residual shear strength ratio (τ_{rl} / σ'_{vo}) for SPT can be determined for Case 1 and Case 2 using Figure 13-36. Note in Figure 13-36 τ_{rl} is equivalent to S_r.

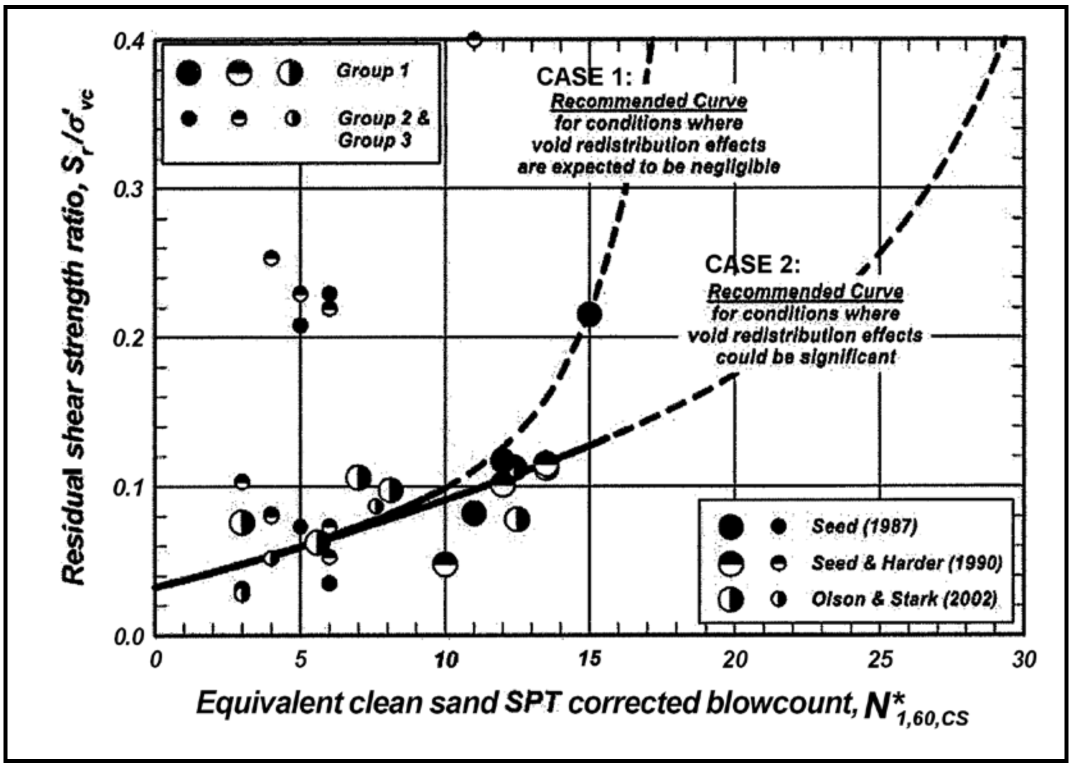


Figure 13-40, Liquefied Shear Strength Ratio - SPT
(Idriss and Boulanger, 2008)

In lieu of using Figure 13-36, the liquefaction residual shear strength ratio (τ_{rl} / σ'_{vo}) for SPT can be determined for Case 1 and Case 2 by using the following equations.

Case 1:

Equation 13-78

$$\left(\frac{\tau_{rl}}{\sigma'_{vo}}\right) = \exp\left[\left(\frac{N^*_{1,60,CS}}{16}\right) + \left(\frac{N^*_{1,60,CS} - 16}{21.2}\right)^3 - 3.0\right] \times \left(1 + \exp\left[\frac{N^*_{1,60,CS}}{2.4} - 6.6\right]\right) \leq \tan\phi'$$

Case 2:

Equation 13-79

$$\left(\frac{\tau_{rl}}{\sigma'_{vo}}\right) = \exp\left[\frac{N^*_{1,60,CS}}{16} + \left(\frac{N^*_{1,60,CS} - 16}{21.2}\right)^3 - 3.0\right] \leq \tan\phi'$$

The CPT correlation uses a corrected normalized and adjusted for fines content CPT tip resistance, $q_{c,1,N,CS}$, computed as shown in Equation 13-73. Corrected and normalized blow counts, $q_{c,1,N}$, should be computed in accordance with Section 13.11.1.4. Values for $\Delta q_{c,1,N\text{-}rl}$ can be found in Table 13-7.

Equation 13-80

$$q_{c,1,N,CS} = q_{c,1,N} + \Delta q_{c,1,N-rl}$$

Table 13-7, Values of $\Delta q_{c,1,N,\text{-}rl}$
(Idriss and Boulanger, 2008)

Fines Content, FC (% passing No. 200 sieve)	$\Delta q_{c,1,N\text{-}rl}$
10	10
25	25
50	45
75	55

The liquefaction residual shear strength ratio (τ_{rl} / σ'_{vo}) for CPT can be determined for Case 1 and Case 2 using Figure 13-37.

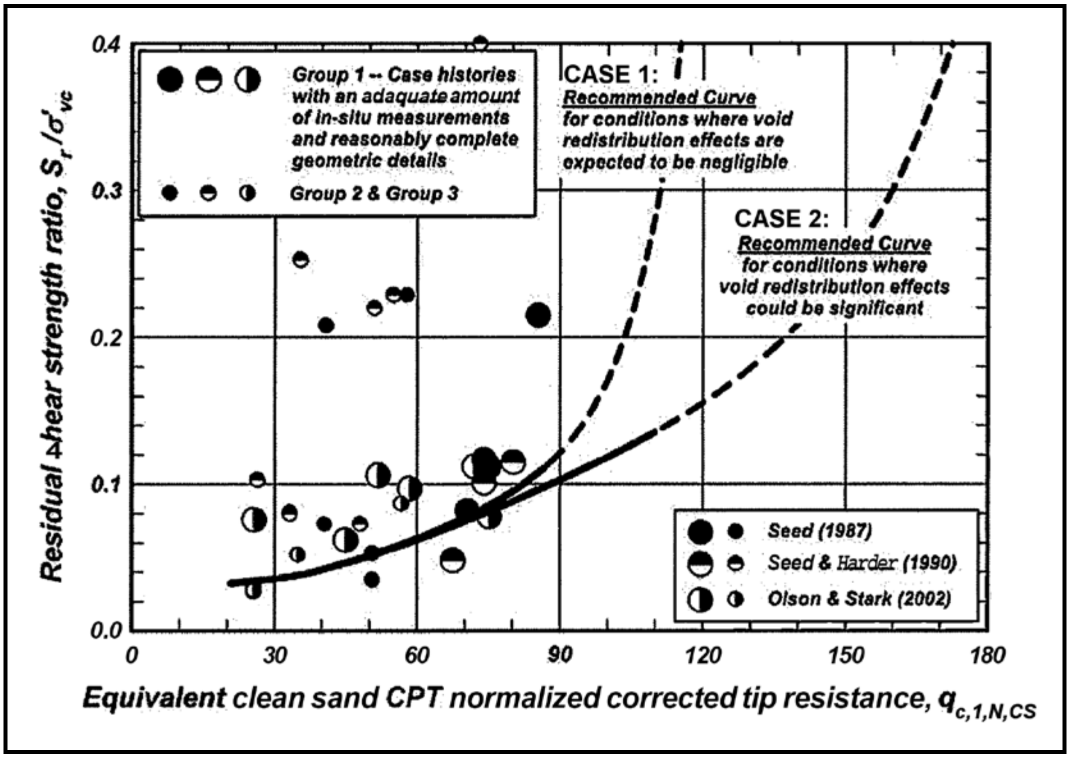


Figure 13-41, Liquefied Shear Strength Ratio - CPT Tip Resistance (Idriss and Boulanger, 2008)

In lieu of using Figure 13-37, the liquefaction residual shear strength ratio (τ_{rl} / σ'_{vo}) for CPT can be determined for Case 1 and Case 2 by using the following equations.

Case 1:

Equation 13-81

$$\left(\frac{\tau_{rl}}{\sigma'_{vo}}\right)=\exp\left[\left(\frac{q_{c,1,N,CS}}{24.5}\right)-\left(\frac{q_{c,1,N,CS}}{61.7}\right)^{2}+\left(\frac{q_{c,1,N,CS}}{106}\right)^{3}-4.42\right]\leq\tan\phi'$$

Case 2:

Equation 13-82

$$\left(\frac{\tau_{rl}}{\sigma'_{vo}}\right)=\exp\left[\left(\frac{q_{c,1,N,CS}}{24.5}\right)-\left(\frac{q_{c,1,N,CS}}{61.7}\right)^{2}+\left(\frac{q_{c,1,N,CS}}{106}\right)^{3}-4.42\right]\times\left(1+\exp\left[\left(\frac{q_{c,1,N,CS}}{11.1}\right)-9.82\right]\right)\leq\tan\phi'$$

13.12.3 Clay-Like Soil Cyclic Shear Strength Triggering

NS Clay-Like soils with soil SSL triggering resistance ratio $(D/C)_{SL}$ greater than the Clay-Like soil SSL triggering resistance factor ($\varphi_{SL\text{-}Clay}$) will be subject to soil SSL approximately equal to the cyclic softening residual shear strength, τ_{rs}. The resistance factor (φ) indicated in Figure 13-38 are for illustration purposes. Resistance factors used for design shall be those presented in Chapter 9.

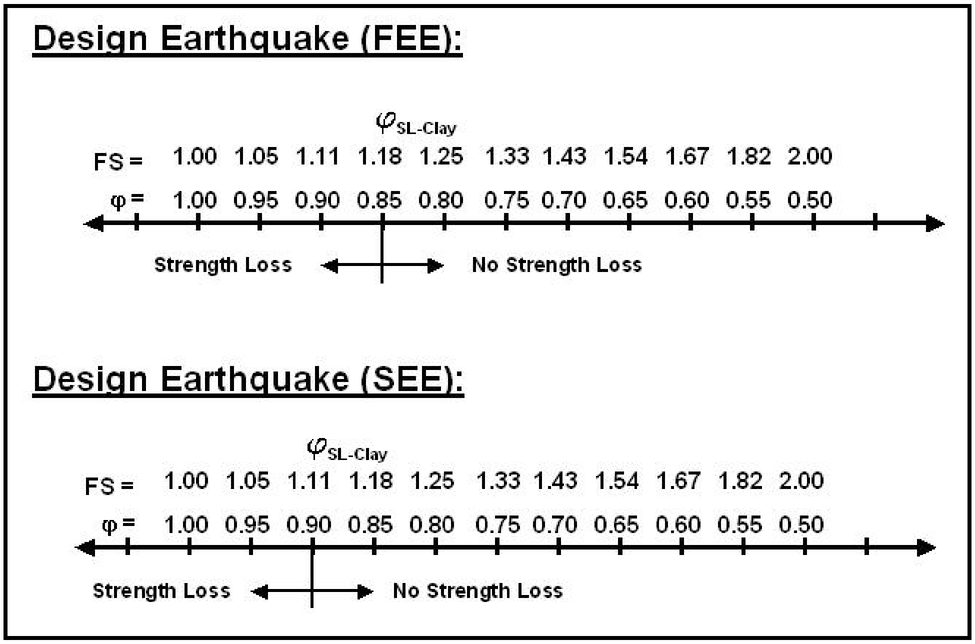


Figure 13-42, Shear Strength of Clay-Like Soils

13.12.4 Clay-Like Soil Cyclic Softening Shear Strength

NS Clay-Like soils below the water table having low sensitivity ($S_t < 5$) and subjected to modest cyclic shear stresses can produce significant permanent strains that can lead to stresses near the soil's yield stress. The residual cyclic softening shear strength, τ_{rs}, of cohesive soils can be estimated by reducing the soil's undrained shear strength ($\tau_{Peak} = S_u$) using the following equation.

$$\tau_{rs} = 0.8\tau_{Peak} \qquad \text{Equation 13-83}$$

Highly sensitive Clay-Like soils having sensitivity ratio, $S_t \geq 5$ that are subject to modest cyclic shear stresses can experience moderate to significant loss in soil shear strengths. The reduced shear strength can be estimated by determining the remolded soil shear strength ($\tau_{remolded}$) as indicated in Chapter 7.

13.12.5 Seismic Soil Shear Strength Selection

The use of drained/undrained soil shear strengths is dependent on the type of soil and the shear strain level the soil is experiencing. Large variations in shear strain levels can occur during a seismic event from small strains during cyclic loadings to large strains during soil failures. The Extreme Event I limit state is used to perform geotechnical analyses for earthquake loadings (Design Earthquake Events FEE and SEE). Because performance limits for the Extreme Event I limit state allow for deformations, the selected shear strength will depend on the strain level that the soil will experience and its potential for soil SSL.

Soil shear strength selection for seismic analyses should be made based on laboratory testing and soil strain level anticipated from analyses. Table 13-8 provides a summary of "general" soil behavior (shear stress vs. strain) observed from published soil stress-strain curves from Holtz and Kovacs (1981), Terzaghi, Peck, and Mesri (1996), and Duncan and Wright (2005). Table 13-8 should be used for "general" guidance on the selection of seismic shear strengths based on soil type and soil strain level anticipated from analyses.

Table 13-8, Seismic Soil Shear Strength Selection

Sand-Like Soils (Undrained)	**Strain Level at Failure** (1)			
	Cyclic Strains	**±5% Strains**	**15–20% Strains**	**Large Strains >20%**
Med. To Dense Sand	τ_{Peak}	τ_{Peak}	τ_{r}	τ_{r}
Non-Liquefying Loose Sands	τ_{Peak}	τ_{Peak}	τ_{Peak}	τ_{Peak}
Liquefied Soils	τ_{rl}	τ_{rl}	τ_{rl}	τ_{rl}
NS Clay-Like Soils (Undrained)	**Strain Level at Failure** (1)			
	Cyclic Strains	**±2% Strains**	**10–15% Strains**	**Large Strains >15%**
OCR =1, $S_t < 5$	τ_{rs}	τ_{Peak}	τ_{Peak}	τ_{Peak}
OCR >1, $S_t < 5$	τ_{rs}	τ_{Peak}	τ_{r}	τ_{r}
HS Clay-Like Soils (Undrained)	**Strain Level at Failure** (1)			
	Typically Failure < 3%			
Highly Sensitive ($S_t \geq 5$)	$\tau_{remolded}$			

Shear Strength Nomenclature:

τ_{Peak} = Peak Soil Shear Strength

τ_{r} = Residual Soil Shear Strength

$\tau_{remolded}$ = Remolded Soil Shear Strength

τ_{rl} = Cyclic Liquefaction Residual Shear Strength

τ_{rs} = Cyclic Softening Residual 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.

13.13 FLOW SLIDE FAILURE

Flow liquefaction failure occurs when the soils exhibit strain softening and gravitational shear stresses are larger than the cyclic liquefaction residual soil shear strength. The strain softening can be a result of monotonic or cyclic undrained loading. Flow liquefaction failures typically occur rapidly and are usually catastrophic. Earthquake-induced flow liquefaction tends to occur after the cyclic loading ceases due to the progressive nature of the load redistribution; however, if the soils are sufficiently loose and the static shear stresses are sufficiently large, the earthquake loading may trigger essentially “spontaneous liquefaction” within the first few cycles of loading.

Flow failures are the most destructive form of liquefaction-induced ground failure. A flow failure is characterized by substantial masses of surficial soils undergoing large translational or rotational deformations and typically occurs after the earthquake shaking has ceased. The surficial soils undergo deformations when static driving forces exceed the average soil shearing stress along the critical failure surface. The soil shearing stress along the critical shear surface is significantly reduced due to the strain softening of underlying granular loose deposits that occurs during flow liquefaction. Because flow liquefaction tends to occur after the cyclic loading ceases, the effects of earthquake ground motions (inertial forces due to the earthquake shaking) are not included in the analyses.

The evaluation of the onset of a flow failure proceeds as follows:

1. Use steeply sloped ground site conditions to evaluate the potential for flow failure

2. If flow failure potential exists, perform a SSL Triggering analysis to determine which soils are susceptible to SSL.

3. Assign appropriate soil shear strengths to Sand-Like, NS Clay-Like, and HS Clay-Like soils susceptible to SSL and undrained/drained shear strengths to soils not susceptible to SSL.

4. Perform a conventional static slope stability analysis (no seismic acceleration coefficient). Determine the static resistance to flow failure $(D/C)_{Flow}$ and the required resistance factor against flow failure, φ_{Flow}. If the static resistance to flow failure $(D/C)_{Flow} > \varphi_{Flow}$, flow failure potential exists at the site. The LRFD equation for use in determining the onset of flow failure is shown below.

$$\left(\frac{D}{C}\right)_{Flow} \leq \varphi_{Flow} \qquad \text{Equation 13-84}$$

The magnitude of flow failure deformations is typically in excess of 25 feet, depending on the geometry of the flowing ground, extent of strain softening of the subsurface soils, and the soil stratification. Estimation of lateral flow deformation is very complex and there are currently no accepted methods for evaluating this type of deformation. Since flow failure deformations can not be reliably estimated, it is assumed that the soils will undergo unlimited deformation and as a result exert soil pressure loads on any structures that are affected by the flow failure movements.

13.14 LATERAL SPREAD

Lateral spreading is the horizontal displacement that occurs on mostly level ground or gentle slopes (< 5 degrees) as a result of cyclic liquefaction of shallow Sand-Like soil deposits. The soil can slide as intact blocks down the slope towards a free face such as an incised river channel. Case studies have reported displacements as great as 30 feet with smaller associated settlements.

Lateral spread occurs as a result of the static and dynamic (earthquake shaking) driving forces applied to surficial soils that have experienced or undergone significant reduction in soil shear strength due to a liquefaction of a loose sand layer typically located within the upper 10 feet. The main distinction between lateral spread and general seismic slope instability is that lateral spread typically occurs on level ground sites not susceptible to flow slide failure and soil SSL along the failure surface are generally greater than 20% when compared to the shear strengths along the failure surface prior to the soil SSL. As a result of the slope instability, a failure surface resembling a sliding block typically develops along the liquefied soils and is subject to lateral displacements until equilibrium is restored. In addition to lateral spread deformations, the ground is also susceptible to seismic settlement. A liquefaction potential assessment in accordance with Sections 13.6 and 13.7 should first be made to determine if screening and triggering of lateral spread is possible. Since cyclic soil strength degradation or liquefaction may occur during the earthquake shaking, the effects of earthquake ground motions (inertial forces) and residual shear strengths (liquefied soils and non-liquefied soils) shall be used during the evaluation of the potential for lateral spread.

The lateral spread displacement evaluation process is as follows:

Step 1. Perform a SSL Triggering analysis for level ground sites to determine which soils are susceptible to SSL.

Step 2. Assign appropriate soil shear strengths to Sand-Like, NS Clay-Like, and HS Clay-Like soils susceptible to SSL and undrained/drained shear strengths to soils not susceptible to SSL.

Step 3. Perform a conventional pseudo-static slope stability analysis with average horizontal acceleration coefficient with adjustments for wave scattering (k_h). Determine the static resistance ratio to lateral spread $(D/C)_{Spread}$ and the required resistance factor against lateral spread, φ_{Spread}. If the static resistance ratio to lateral spread $(D/C)_{Spread} > \varphi_{Spread}$, lateral spread potential exists at the site. The LRFD equation used in determining the onset of lateral spread is shown below.

$$\left(\frac{D}{C}\right)_{Spread} \leq \varphi_{Spread} \qquad \text{Equation 13-85}$$

Step 4. Perform empirical or semi-empirical deformation analyses as described in this Section. If the computed deformations multiplied by a factor of "2" is less than the performance limit requirements in Chapter 10 (2 x deformation computed ≤ required performance limits) then no deformation analysis is required. A factor of "2" times the displacements computed has been selected because of the associated uncertainty inherent in the empirical or semi-empirical deformation analytical methods.

Step 5. If the displacements computed in Step 4 exceed the performance limits, additional displacement calculations should be performed using all of the empirical and semi-empirical methods for computing lateral spread deformations that are presented in this section. The Newmark displacement methods may be used with caution provided that the requirements stated below are adhered to.

Step 6. From the displacement computations performed in Steps 4 and 5, the designer will need to evaluate if there are any trends in the displacements computed. It should be noted that these displacement methods have been shown by various case histories to be very unreliable with just as many under predictions as over predictions, up to a factor of "2" as was used in Step 4.

Empirical/semi-empirical methods have been developed based on case history databases for specific types of failure modes, earthquake characteristics, site geometry, and subsurface soils. Methods indicated as semi-empirical are based on some numerical basis to estimate residual shear strains in the soils after liquefaction. Empirical and semi-empirical methods are only used for screening purposes because of the low reliability of the predictions. The following empirical and semi-empirical methods have been developed specifically to evaluate lateral spread:

- Youd et al., 2002 – Multilinear Regression Equations for Prediction of Lateral Spread Displacements
- Rauch and Martin, 2000- EPOLLS Model for Predicting Average Displacements on Lateral Spreads
- Zhang et al., 2004 – SPT or CPT Liquefaction-Induced Lateral Displacements

Newmark methods of displacement analysis are typically not appropriate for evaluation of lateral spreads when soil SSL are greater than 50% when compared to the shear strengths prior to the SSL. If it can be shown that the shear failure surface as a whole has not lost more than 50% of the shear failure resistance prior to the soils SSL, the Newmark method may be used. The Newmark method may also be used without adjusting soil shear strengths to obtain a lower bound displacement. If the lower bound displacement exceeds the performance limits mitigation of the hazard will be required. Newmark methods described in Section 13.17 should be used with caution.

13.14.1 Multilinear Regression of Lateral Spread Displacements

Youd et al. (2002) revised the multilinear regression empirical equations previously developed by Bartlett and Youd (1992, 1995) for prediction of lateral spread displacement. This method used a large database of lateral spread case histories from past earthquakes. A review of the Bartlett and Youd, 1992, case history database reveals measured displacements that range from less than one inch to as much as 30 feet. The predicted displacements vary from one-half of the measured value to twice the measured value. Because of the high uncertainty associated with lateral displacement values computed with this method it is only used for screening of potential for lateral spread displacements. It should be noted that the following equations are valid when liquefaction occurs over a widespread area, and not just isolated pockets. Liquefiable soils layers must be identified as indicated in Sections 13.6 and 13.7. Guidance for specific input of each of the parameters in the following equations is provided by Youd, et al. (2002).

For ground slope condition (Figure 13-40: Case 1):

Equation 13-86

$$\log D_H = -16.213 + 1.532\,M_w - 1.406\log(R^{*}) - 0.012R + 0.338\log(S) + 0.540\log T_{15} + 3.413\log(100 - F_{15}) - 0.795\log(D50_{15} + 0.1\text{ mm})$$

For free-face condition (Figure 13-40 Case 2):

Equation 13-87

$$\log D_H = -16.713 + 1.532\,M_w - 1.406\log(R^{*}) - 0.012R + 0.592\log(W) + 0.540\log(T_{15}) + 3.413\log(100 - F_{15}) - 0.795\log(D50_{15} + 0.1\text{ mm})$$

Where:

D_H = Estimated lateral ground displacement in meters

T_{15} = Cumulative thickness of saturated granular layers with corrected blow counts, $(N_1)_{60}$, less than or equal to 15, in meters.

$D50_{15}$ = Average mean grain size in granular layer included in T_{15} in mm.

F_{15} = Average fines content (passing #200 sieve) for granular layers included in T_{15} in percent.

M_w = Design earthquake magnitude (moment magnitude).

R = Site-to-source distance, in kilometers.

R_o = Distance factor that is a function of earthquake magnitude, M_w and is computed using the following equation.

$$\boldsymbol{R_o = 10^{(0.89 M_w - 5.64)}}$$ Equation 13-88

R^* = Modified source distance and is computed using the following equation.

$$\boldsymbol{R^* = R + R_o}$$ Equation 13-89

S = Ground slope, in percent.

W = Ratio of the height (H) of the free face to the distance (L) from the base of the free face to the point in question, in percent (i.e.,100H/L).

The design earthquake moment magnitude, M_w, and the site-to-source distance, R, shall be those determined for the design earthquake under evaluation as required in Chapter 12.

13.14.2 EPOLLS – Average Horizontal Lateral Spread Displacements

Rauch and Martin (2000) developed the EPOLLS (Empirical Prediction Of Liquefaction-induced Lateral Spreading) model to predict the average horizontal ground surface displacements due to liquefaction-induced lateral spread. The liquefaction accounts only for the SSL of Sand-Like soils. This method does not account for SSL in NS Clay-Like soils or HS Clay-Like soils. The EPOLLS model was developed from a multiple regression analysis of data from 71 lateral spread case studies with 15 earthquakes in western North America and eastern Asia (mostly California, Alaska, and Japan). The maximum average horizontal displacements in the database is approximately 16 feet (4.8 m) with approximately 1/3 of the cases having an average horizontal displacement less than 8.5 feet (2.5 m). The EPOLLS model uses seismic input, site topography, and subsurface conditions to predict horizontal ground surface displacements.

The EPOLLS model predicts the average displacement across the surface of a lateral spread. The EPOLLS model was developed based on three components: Regional, Site, and Geotechnical. The Regional component (R-EPOLLS) accounts for earthquake source factors affecting the magnitude of the lateral spread such as the earthquake magnitude, distance to fault rupture, peak horizontal acceleration, and duration of strong shaking. The Site component (S-EPOLLS) accounts for site factors affecting the magnitude of the lateral spread such as the length of sliding area, slope of the ground surface, and height of free-face. The Geotechnical component (G-EPOLLS) accounts for geotechnical factors affecting the magnitude of the lateral spread such as depth of liquefaction zone and depth to the top of the liquefied soils. It should be noted that the following equations are valid when cyclic liquefaction of Sand-Like soils occurs over a widespread area, and not just isolated pockets. Liquefiable soils layers must be identified as indicated in Sections 13.6 and 13.7. Guidance for specific input of each of the parameters in the following equations is provided by Rauch and Martin (2000).

<u>R-EPOLLS (Regional Component)</u>:

$$D_R = (613M_W - 13.9R - 2420A_{max} - 11.4D_{5\text{-}95})/1000 \qquad \text{Equation 13-90}$$

$$\text{R-EPOLLS Avg. Displacement} = (D_R - 2.21)^2 + 0.149 \qquad \text{Equation 13-91}$$

<u>S-EPOLLS (Site Component):</u>

$$D_S = 0.523L_{slide} + 42.3S_{top} + 31.3H_{face})/1000 \qquad \text{Equation 13-92}$$

$$\text{S-EPOLLS Avg. Displacement} = (D_R + D_S - 2.44)^2 + 0.111 \qquad \text{Equation 13-93}$$

<u>G-EPOLLS (Geotechnical Component):</u>

$$D_G = (50.6Z_{Bottom} - 86.1Z_{Top})/1000 \qquad \text{Equation 13-94}$$

$$\text{G-EPOLLS Avg. Displacement} = (D_R + D_S + D_G - 2.49)^2 + 0.124 \qquad \text{Equation 13-95}$$

Where:

M_w = Design earthquake moment magnitude

R = Site-to-source, in kilometers

A_{max} = Peak horizontal acceleration, units of g (gravity)

$D_{5\text{-}95}$ = Duration of strong shaking, units of seconds

L_{Slide} = Length of sliding area, in meters

S_{Top} = Slope of ground surface, in units of percent (%)

H_{Face} = Height of free-face, in units meters

Z_{Bottom} = Depth to bottom of liquefied layer from ground surface, in units of meters.

Z_{Top} = Depth to top of liquefied layer from ground surface, in units of meters.

The design earthquake moment magnitude (M_w), source to site distance (R), peak horizontal acceleration (A_{max} = PGA), and the duration of strong shaking ($D_{5\text{-}95}$) are determined for the design earthquake under evaluation as indicated in Chapter 12.

The model parameters used in the equation should be checked to ensure that a prediction is within the limits of the data indicated in Table 13-9 that was used to develop the model.

The G-EPOLLS Avg. Displacement results should be used in Step 4 (Section 13.14) of the lateral spread displacement evaluation process.

Table 13-9, Limiting Range of EPOLLS Model Parameters (Rauch and Martin, 2000 with permission from ASCE)

Variables	Minimum Value	Maximum Value	Units
M_w	6.5	9.2	---
R	0	119	Km
A_{max}	.16	0.52	G
$D_{5\text{-}95}$	4	88	Sec
L_{Slide}	20	1,360	m
S_{Top}	-0.7	5.2	%
H_{Face}	0	9.0	m
Z_{Bottom}	2.4	12.4	m
Z_{Top}	0.9	7.3	m

A check to ensure that parameter combinations do not exceed the limits of the database should be made by computing D_R, (D_R + D_S), and (D_R + D_S + D_G) are within the limits of Table 13-10. Note that the average horizontal lateral spread displacement computed using G-EPOLLS ranges from 0.23 to 4.29 meters.

Table 13-10, Limiting Range of EPOLLS Variables
(Rauch and Martin, 2000 with permission from ASCE)

Variables	Minimum Value	Maximum Value	Units
D_R	6.5	9.2	m
$(D_R + D_S)$	0	119	m
$(D_R + D_S + D_G)$	.16	0.52	m
R-EPOLLS Avg. Displacement	4	88	m
S-EPOLLS Avg. Displacement	20	1,360	m
G-EPOLLS Avg. Displacement	-0.7	5.2	m

13.14.3 SPT/CPT Liquefaction-Induced Lateral Displacements

Liquefaction-induced lateral spread displacements can be estimated by either SPT or CPT in-situ testing results using the method proposed by Zhang et al., 2004. The SPT and CPT normalized values of $N^*_{1,60,cs}$ and $q_{c,1,N,cs}$ are computed in accordance with Sections 13.11.2 and 13.11.3, respectively. The relationship between the maximum cyclic shear strain, γ_{max}, and the liquefaction triggering resistance ratio, $(D/C)_{SL}$, for soils with different relative densities, D_r, is provided in Figure 13-39. The liquefaction triggering resistance ratio, $(D/C)_{SL}$, is obtained from the soil SSL triggering analysis for level ground sites in Section 13.7.

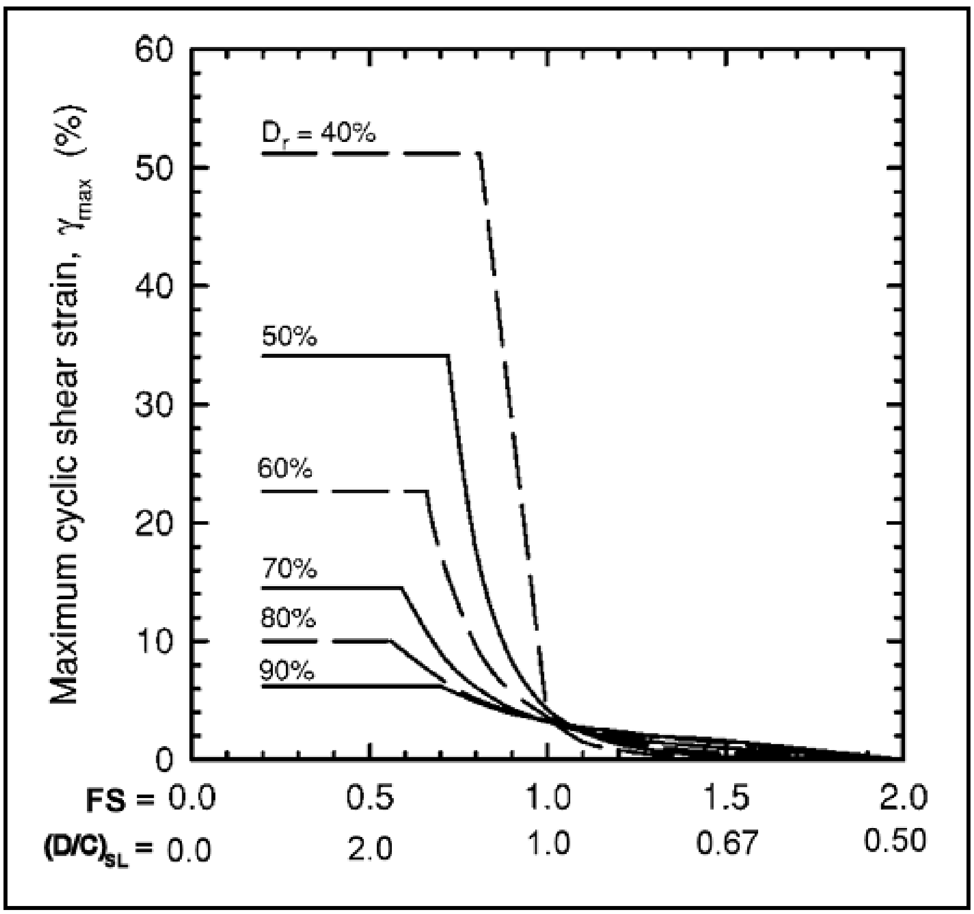


Figure 13-43, Relationship Between Maximum Cyclic Shear Strain and φ
(Zhang et al., 2004 with permission from ASCE)

The SPT and CPT equivalent clean sand normalized values of $N^*_{1,60,cs}$ and $q_{c,1,N,cs}$ can be used to estimate the relative density, D_r, of the potentially liquefiable soils. The relative density, D_r, can be estimated from SPT blow counts ($N^*_{1,60,cs}$ in units of blows/foot) using the following equation.

Equation 13-96

$$D_r = 14 \cdot \sqrt{N^*_{1,60,cs}} \qquad \left[N^*_{1,60,cs} \leq 42\ blows/foot\right]$$

The relative density, D_r, can be estimated from CPT tip resistance ($q_{1,c,mod}$ in units of kPa) using the following equation.

Equation 13-97

$$D_r = -85 + 76\log\left(q_{c,1,N,cs}\right) \qquad \left[q_{c,1,N,cs} \leq 200\right]$$

In lieu of using Figure 13-39 to estimate the maximum cyclic shear strain, γ_{max}, the equations listed in Table 13-11 can be used.

Table 13-11, Relationship Between Maximum Cyclic Shear Strain and φ[1]
(Zhang et al., 2004 – data from Ishihara and Yoshimine, 1992 and Seed, 1979)

$$\text{If } D_r = 90\%, \quad \gamma_{\max} = 3.26\left(\frac{1}{\varphi}\right)^{-1.80} \quad \text{for } 0.70 \le \left(\frac{1}{\varphi}\right) \le 2.00$$

$$\text{If } D_r = 90\%, \quad \gamma_{\max} = 6.2 \quad \text{for } \left(\frac{1}{\varphi}\right) \le 0.70$$

$$\text{If } D_r = 80\%, \quad \gamma_{\max} = 3.22\left(\frac{1}{\varphi}\right)^{-2.08} \quad \text{for } 0.56 \le \left(\frac{1}{\varphi}\right) \le 2.00$$

$$\text{If } D_r = 80\%, \quad \gamma_{\max} = 10.0 \quad \text{for } \left(\frac{1}{\varphi}\right) \le 0.56$$

$$\text{If } D_r = 70\%, \quad \gamma_{\max} = 3.20\left(\frac{1}{\varphi}\right)^{-2.89} \quad \text{for } 0.59 \le \left(\frac{1}{\varphi}\right) \le 2.00$$

$$\text{If } D_r = 70\%, \quad \gamma_{\max} = 14.5 \quad \text{for } \left(\frac{1}{\varphi}\right) \le 0.59$$

$$\text{If } D_r = 60\%, \quad \gamma_{\max} = 3.58\left(\frac{1}{\varphi}\right)^{-4.42} \quad \text{for } 0.66 \le \left(\frac{1}{\varphi}\right) \le 2.00$$

$$\text{If } D_r = 60\%, \quad \gamma_{\max} = 22.7 \quad \text{for } \left(\frac{1}{\varphi}\right) \le 0.66$$

$$\text{If } D_r = 50\%, \quad \gamma_{\max} = 4.22\left(\frac{1}{\varphi}\right)^{-6.39} \quad \text{for } 0.72 \le \left(\frac{1}{\varphi}\right) \le 2.00$$

$$\text{If } D_r = 50\%, \quad \gamma_{\max} = 34.1 \quad \text{for } \left(\frac{1}{\varphi}\right) \le 0.72$$

$$\text{If } D_r = 40\%, \quad \gamma_{\max} = 3.31\left(\frac{1}{\varphi}\right)^{-7.97} \quad \text{for } 1.00 \le \left(\frac{1}{\varphi}\right) \le 2.00$$

$$\text{If } D_r = 40\%, \quad \gamma_{\max} = 250\left(1.0 - \left(\frac{1}{\varphi}\right)\right) + 3.5 \quad \text{for } 0.81 \le \left(\frac{1}{\varphi}\right) \le 1.00$$

$$\text{If } D_r = 40\%, \quad \gamma_{\max} = 51.2 \quad \text{for } \left(\frac{1}{\varphi}\right) \le 0.81$$

[1] $\varphi = (D/C)_{SL}$

A Lateral Displacement Index (LDI) has been developed in order to quantify the potential for lateral displacements for a given soil profile, soil properties, and earthquake characteristics. The LDI is computed by integrating the calculated maximum cyclic shear strain, γ_{max}, over the maximum depth of all potential liquefiable layers. The LDI can be computed as indicated in the following equation.

$$LDI = \int_{0}^{Z_{max}} \gamma_{max} dz \qquad \text{Equation 13-98}$$

Where Z_{max} is the maximum depth below all the potential liquefiable layers with a calculated liquefaction resistance ratio, $(D/C)_{SL} < 0.50$.

The magnitude of lateral displacement, LD, is dependent on the LDI and the geometric parameters characterizing ground geometry. The following ground geometry cases are used in these analyses.

Case 1. Gently sloping ground without a free face

Case 2. Nearly level ground with a free face

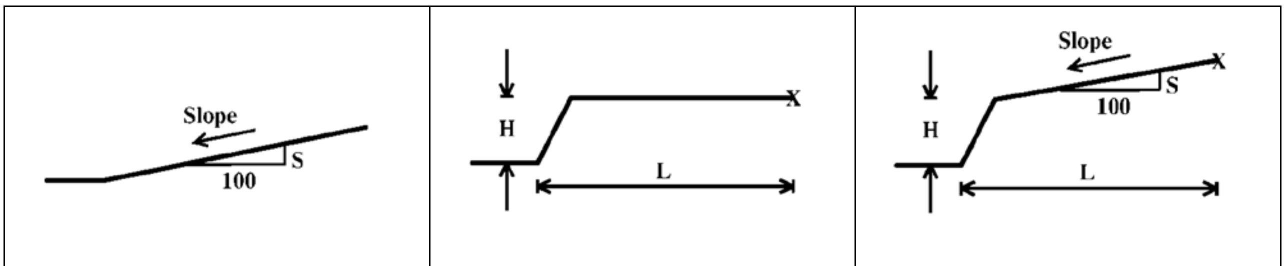
Case 3. Gently sloping ground with a free face

The ground geometry parameters used to define the above cases are described below and illustrated in Figure 13-40.

S = Ground Slope (%)

H = Free Face Height (Units consistent with units of L)

L = Distance to Free Face (Units consistent with units of H)



Case 1: Gently Sloping Ground Without Free Face	Case 2: Nearly Level Ground With Free Face	Case 3: Gently Sloping Ground With Free Face

Figure 13-44, Ground Geometry Cases
(Zhang et al., 2004 with permission from ASCE)

Once the ground geometry has been defined, the lateral displacements, LD, can be computed as indicated below:

Case 1: Gently Sloping Ground Without Free Face

Equation 13-99

$$LD = LDI(S + 0.2) \qquad [0.20\% < S < 3.5\%]$$

Cases 2 or 3: Nearly Level Ground to Gently Sloping Ground With Free Face

Equation 13-100

$$\boldsymbol{LD = 6 \cdot LDI(L/H)^{-0.8}} \qquad \boldsymbol{[4 < (L/H) < 40]}$$

Since this method is a semi-empirical method, it should be limited to the ranges of earthquake properties and ground conditions included within the case history database as indicated below.

- Moment Magnitude, M_w: $6.4 \le M_w \le 9.2$
- Peak Horizontal Ground Acceleration, a_{max}: $0.19g \le a_{max} \le 0.60g$
- Free Face Height, H: H < 60 feet

13.15 SEISMIC GLOBAL STABILITY

Earthquake-induced global instability can occur at bridge abutments, roadway embankments, bridge approach fills, natural cut slopes, and at ERSs. This geotechnical seismic hazard occurs as a coherent sliding soil mass that moves along a critical shear failure surface. The global slope instability is typically slow moving and generally consists of a deep-seated failure surface that is either circular or sliding block shape. The triggering mechanism for these types of slope instabilities is the earthquake horizontal accelerations that induce inertial driving forces in addition to the initial static driving stresses that already exist. As a consequence of the cyclic shear stress induced by the earthquake, some soils may experience strain softening which results in a reduced soil shearing resistance. The reduction in soil shear strength must not be greater than 20%. If this threshold is exceeded, then the geotechnical seismic hazard is considered a lateral spread. The combination of the inertial driving forces and the reduced soil shear strength can result in instability of the earthen slopes. This situation could lead to significant deformations that can affect adjacent transportation structures such as bridges, roadways, and ERSs.

The standard-of-practice for evaluating seismic global instabilities consists of performing pseudo-static slope stability analyses. The pseudo-static slope stability analysis is a modified conventional slope stability analysis that allows the inclusion of inertial driving forces generated by the earthquake as equivalent static horizontal force acting on the potential sliding mass. The pseudo-static slope stability method uses the average horizontal acceleration coefficients adjusted for wave scattering (k_h) as indicated in Section 13.16 to compute the inertial loadings in the seismic global stability analysis. If the seismic slope stability ratio $(D/C)_{EQ\text{-}Stability} \le \varphi_{EQ\text{-}Stability}$, then the Extreme Event I (EEI) limit state stability requirements and performance limits (no deformations) have been satisfied. If the seismic slope stability ratio $(D/C)_{EQ\text{-}Stability} > \varphi_{EQ\text{-}Stability}$, then a Newmark sliding block analysis is performed to estimate the displacements and determine if they meet the performance limits in Chapter 10. A reduction in the horizontal seismic coefficient (k_h) due to displacements is dependent on the yield acceleration, the average horizontal acceleration coefficients adjusted for wave scattering (k_h), peak horizontal ground acceleration (PGA) at the ground surface, and the peak ground velocity (PGV or V_{Peak}).

The overall seismic slope stability evaluation process is shown in Figure 13-41. The evaluation of the onset of seismic global instability proceeds as follows:

1. Determine earthquake parameters (PGA, S_{D1}, and PGV) from Chapter 12.

2. Determine wave scattering scaling factor (α_w) from Section 13.16.

3. Compute average horizontal seismic coefficient, k_h, in accordance with Section 13.16.

4. Perform a conventional pseudo-static slope stability analysis conforming to the requirements of Chapter 17 with average horizontal acceleration coefficient with adjustments for wave scattering (k_h). The vertical acceleration coefficient (k_v) is assumed to equal zero. Assign appropriate soil shear strengths based on soil SSL triggering to Sand-Like soils, NS Clay-Like soils, and HS Clay-Like soils that are susceptible to soil SSL. Use fully mobilized undrained/drained shear strengths for soils not susceptible to soil SSL.

5. Evaluate the critical slope failure surfaces to determine if any critical failure surfaces intersect soils with potential SSL. If the failure surfaces intersect these soil layers, compute the shear failure surface resistance as a "whole" and compare it to the shear failure resistance prior to the soils SSL. If it can be shown that the shear failure surface as a "whole" has not lost more than 20% strength, then hazard analysis can be continued as a seismic instability; otherwise, analyze the site as a lateral spread geotechnical seismic hazard.

6. Determine the seismic stability resistance ratio $(D/C)_{EQ\text{-}Stability}$. Obtain the required seismic slope instability resistance factor ($\varphi_{EQ\text{-}Stability}$) from Chapter 9. The LRFD equation used to determine the onset of lateral spread is shown below.

$$\left(\frac{\boldsymbol{D}}{\boldsymbol{C}}\right)_{\boldsymbol{EQ-Stability}} \leq \varphi_{\boldsymbol{EQ-Stability}} \qquad \text{Equation 13-101}$$

If the seismic instability ratio $(D/C)_{EQ\text{-}Stability} \leq \varphi_{EQ\text{-}Stability}$, then there is no potential for seismic slope instability hazard and the analysis can be terminated.

If the seismic instability ratio $(D/C)_{EQ\text{-}Stability} > \varphi_{EQ\text{-}Stability}$, seismic instability potential exists at the site, the evaluation process should continue to Step 7 to evaluate the displacements.

7. Compute the horizontal yield acceleration (k_y) by varying the horizontal acceleration until the seismic instability ratio $(D/C)_{EQ\text{-}Stability}$ =1.

8. Compute the deformations (ΔL) of the seismic slope instability using Newmark sliding block method described in Section 13.17.

9. If the displacements are within the acceptable performance limits, the seismic slope stability hazard is acceptable with respect to the Extreme Event I limit state and the performance, limits. If the deformations computed exceed the performance limits then develop methods to mitigate this hazard as indicated in Chapter 14 and then evaluate the seismic global stability hazard again (Step 4).

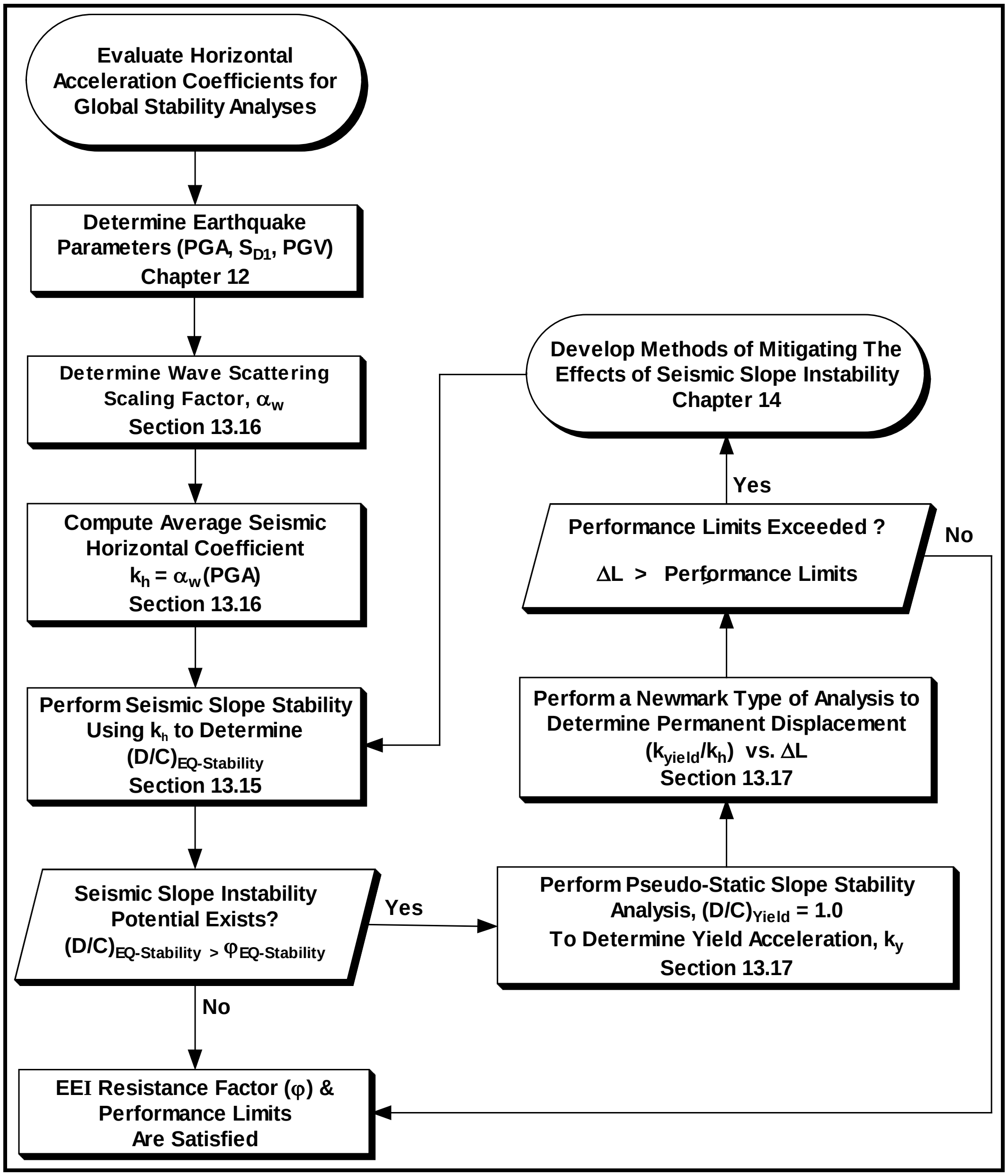


Figure 13-45, Seismic Slope Stability Evaluation Process

Sites that meet any of the following requirements are considered complex sites that may require a site-specific two-dimensional dynamic response analysis to estimate peak ground acceleration at the base of the failure surface, k_{max}. The PCS/GDS should be consulted when a complex site is identified.

- Sites that require a site-specific response analysis
- Geometries such as compound slopes, broken back slopes, etc.
- Soil properties such as sensitive soils, significant liquefaction potential, etc.
- Transportation facilities impacted significantly by failures or deformations

13.16 SEISMIC ACCELERATION COEFFICIENTS

The magnitude of earthquake inertial forces and seismic loading (active / passive pressures) that are used in pseudo-static stability analyses or limit-equilibrium analyses of ERSs are based on computing average horizontal acceleration coefficients adjusted for wave scattering (k_h). The horizontal acceleration coefficient (k_h) is computed by using the peak horizontal acceleration at the ground surface (PGA = a_{max}, etc.) with adjustments that typically reduce the acceleration by taking into account wave scattering of the horizontal ground accelerations and displacements of a yielding structure. The wave scattering scaling factor (α_w) is dependent on the design pseudo-spectral acceleration at 1 second (S_{D1}) from the ADRS curve and the height of the embankment, slope, or ERS.

Seismic inertial loadings are typically estimated by pseudo-static analytical methods that consist of multiplying the average horizontal seismic coefficient (k_h) and average vertical seismic coefficient (k_v) by the mass of the soil or structure that is being accelerated due to the earthquake shaking. The vertical seismic coefficient (k_v) is typically neglected (k_v = 0) by the fact that the higher frequency vertical accelerations will be out of phase with the horizontal accelerations and will have positive and negative contributions to the seismic loadings that on average will tend to cancel itself and therefore, have little, if any effect, on the seismic loading. The horizontal seismic coefficient (k_h) is used to compute a constant horizontal force in global seismic stability of slopes and ERSs. The horizontal seismic coefficient (k_h) has typically been assigned some fraction (0.3 to 0.7) of the peak horizontal ground acceleration (PGA). Reductions in PGA are typically attributable to either wave scattering or stress relief associated with displacements. The displacement dependent stress relief reduction of the horizontal seismic coefficient is computed using the Newmark displacement method as shown in Section 13.17.

Wave scattering is a term used to account for the seismic wave incoherence or variations behind a wall or slope. NCHRP 12-70 (2007) developed a relationship utilizing a scale factor, α_w, (reduction factor) to account for wave scattering as indicated by the following equation.

$$\boldsymbol{k_h} = \boldsymbol{k_{avg}} = \boldsymbol{\alpha_w k}_{\max} \qquad \text{Equation 13-102}$$

Where,

- k_h = Average seismic horizontal coefficient due to wave scattering
- α_w = Wave scattering scaling factor (reduction factor)
- k_{max} = Peak ground acceleration coefficient (PGA) for the design event

The wave scattering scaling factor (α_w) was found to be dependent on the ground motion and the height of the wall or slope as shown in Figure 13-42. For wall or slope heights less than or equal to 20 feet α_w is 1.0. For wall or slope heights greater than 70 feet a numerical analysis is required.

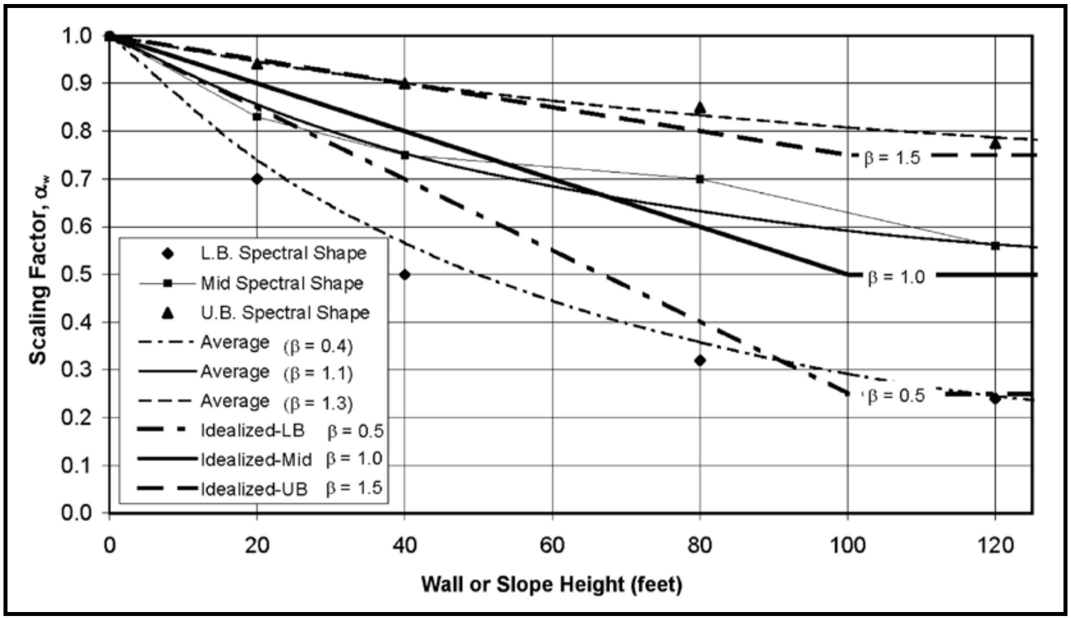


Figure 13-46, Simplified Wave Scattering Scaling Factor (NCHRP 12-70, 2007)

For wall or slope heights greater than 20 feet and less than or equal to 70 feet, α_w shall be determined by the following equation.

$$\alpha_w = 1 + 0.01H[(0.5\beta) - 1] \leq 1.0 \qquad \text{Equation 13-103}$$

Where,

- H = Height of wall or slope (feet)
- β = Ground motion index that is used to characterized shape of the ADRS.

The ground motion index, β, is applicable to a range of $0.5 \leq \beta \leq 1.5$ and is computed using the following equation.

$$\beta = \frac{S_{D1}}{PGA} \qquad \text{Equation 13-104}$$

Where,

- S_{D1} = Peak ADRS spectral acceleration at 1 second (Chapter 12)
- $PSA_{1\text{-}secB\text{-}C}$ = Mapped pseudo spectral acceleration at 1 second (Chapter 11)
- PGA = Peak horizontal acceleration at ground surface (Chapter 12)
- $PGA_{B\text{-}C}$ = Mapped PGA at B/C boundary (Chapter 11)

The wave scattering scaling factor (α_w) is applicable to soil Site Classes C, D, and E. If the Site Class is A or B, the wave scattering scaling factor (α_w) should be increased by 20 percent without exceeding a wave scattering factor of α_w = 1.0 (Use α_w = 1.0 for walls or slopes less than 20 feet in height when founded on Site Class A or B soils).

13.17 NEWMARK SEISMIC DISPLACEMENT METHODS

The Newmark sliding block model is used to evaluate displacements that occur as a result of an imbalance between driving forces (static and seismic) and loss in resisting forces (strain softening of soils) acting on the displaced soil mass. The models that have been developed based on Newmark rigid sliding block assume that the deformation takes place on a well-defined failure surface, the yield acceleration remains constant during shaking, and the soil is perfectly plastic. The displacements are computed based on the cumulative displacements of the sliding mass generated when accelerations exceed the yield acceleration that defines the point of impending displacement. Newmark type methods for computing deformations are typically associated with an improved reliability when compared to the empirical methods because it is a numerical method that permits modeling of the site response for the design earthquake being investigated.

The state-of-practice is that the assumptions used in the Newmark sliding block model provide reasonable results when the shear failure surface as a whole has not lost more than 50% of the shear resistance prior to the soils SSL. These assumptions are applicable to seismic slope instability and to lateral seismic deformation of gravity ERSs that are not significantly affected by cyclic liquefaction. Bardet et al. (1999) observed that when cyclic liquefaction occurs in a lateral spread, the assumptions of the Newmark sliding block model requirements are not met because (1) the shear strain in liquefied soil does not concentrate within a well defined surface, (2) the shear strength and yield acceleration of saturated soils varies during cyclic loading as pore pressure varies, and (3) soils are generally not perfectly plastic materials, but commonly harden and/or soften.

Several analytical methods based on the Newmark sliding block model have been developed to estimate deformations induced by earthquake cyclic loadings. The Newmark type methods typically fall into one of the following categories:

- Newmark Time History Analyses
- Simplified Newmark Charts

The Newmark displacement method can be performed using the design earthquake's time history acceleration record if a site-specific seismic response is performed in accordance with Chapter 12. Alternatively, Simplified Newmark charts can be used when a site-specific seismic response is not performed. The Simplified Newmark charts are based on a large database of earthquake records and the Newmark Time History Analysis method to develop charts that relate the ratio of acceleration to yield acceleration occurring at the base of the sliding mass to ground displacement.

If a site-specific response analysis is performed in accordance with Chapter 12, then the Newmark Time History Analyses should be performed in combination with the Simplified Newmark evaluation to validate deformation analyses performed using the Newmark Time History analyses. If a simplified site response method is used (i.e. three-point ADRS curves) to evaluate the local response site effects, then the Simplified Newmark charts should be used. The Newmark time history method and the Simplified Newmark charts are described in the following Sections.

13.17.1 Newmark Time History Analyses

The Newmark "sliding block" method for analyzing ground displacements along a shear plane was developed by Newmark (1965). Newmark's method has been applied to seismic slope stability performance of dams, embankments, natural slopes, and retaining walls (Newmark, 1965, Makdisi and Seed, 1978, Yegian et al., 1991, Jibson, 1994, and Richard and Elms, 1979). This method is typically incorporated into computer programs as described by Houston et al. (1987).

Randall W. Jibson and Matthew W. Jibson have developed a computer program to model slope performance during earthquakes. The Java program uses a modification of Newmark's method where a decoupled analysis is performed that allows modeling landslides that are not assumed to be rigid blocks. The software and more information can be obtained at the USGS website http://earthquake.usgs.gov/resources/software/slope_perf.php.

Ebeling, et al., (2007) have developed a computer program for the US Army Corps of Engineers that estimates the translational response of retaining walls to earthquake ground motions called $C_{orps}W_{all}$Slip (CWSlip) that could be used to estimate lateral displacements for gravity ERSs.

In this method, the deformations are assumed to occur along a well-defined plane and the sliding mass is assumed to be a rigid block as shown in Figure 13-43. When the earthquake accelerations exceed a yield acceleration threshold, the sliding mass displaces as indicated in Figure 13-44. The displacement accumulates over a time span ($t_3 - t_1$) where the acceleration exceeds the yield acceleration (k_y) at time t_1 to when the induced velocity drops to zero at time t_2. The displacements are computed by double integrating the accelerogram over the time span (t_3 - t_1) cumulative over each cycle for the duration of the earthquake as indicated in Figure 13-44. The total displacement is computed as the cumulative displacement that occurs during the earthquake shaking.

Note that the yield acceleration, k_y, in Figure 13-44 varies with the level of acceleration as a result of the cyclic soil strength degradation or liquefaction. Soils that are subject to significant strain softening will develop lower yield acceleration, k_y, thresholds as the earthquake-induced cyclic soil strength degradation progresses. The yield acceleration, k_y, is generally maintained constant because of the conservative approach used to determine the yield acceleration, k_y.

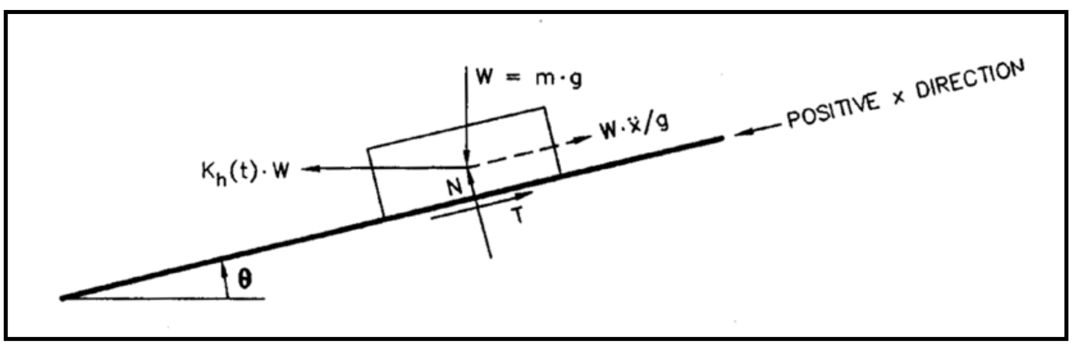


Figure 13-47, Newmark Sliding Block Method (Matasovic, Kavazanjian, and Giroud 1998)

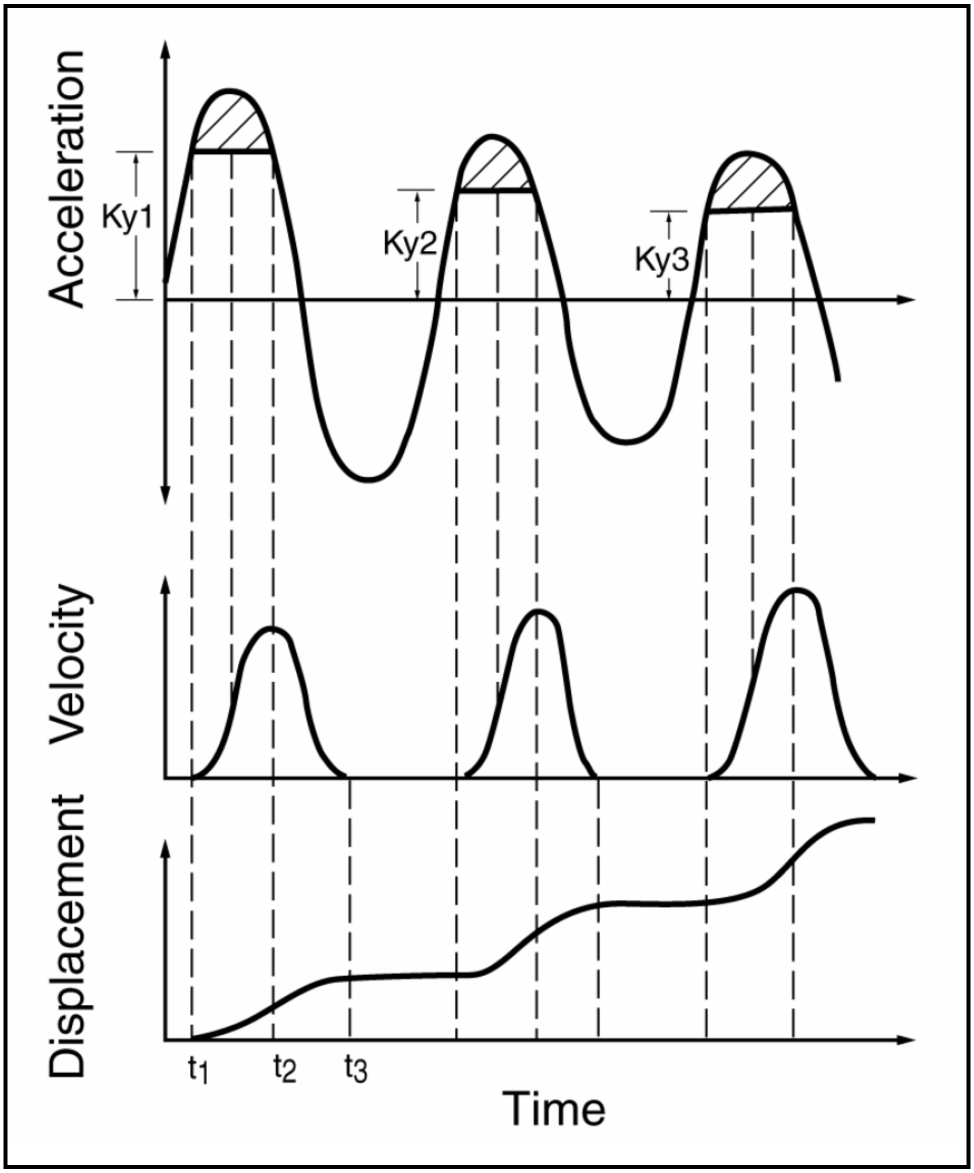


Figure 13-48, Newmark Time History Analysis
(Goodman and Seed, 1966 with permission from ASCE)

The earthquake shaking that triggers the displacement is characterized by an acceleration record at the base of the sliding mass for the design earthquake being evaluated. A minimum of twelve independent earthquake records should be selected from a catalogue of earthquake records that are representative of the source mechanism, magnitude (M_w), and site-to-source distance (R). A sensitivity analysis of the input parameters used in the site-specific response analysis should be performed to evaluate its effect on the magnitude of the displacement computed.

A pseudo-static slope stability analysis is performed to determine the threshold yield acceleration, k_y, where displacements begin to occur for a specific critical failure surface. The pseudo-static slope stability analysis should be performed with cyclic residual shear strength (Section 13.12) assigned to soils with the potential for soil SSL. The yield acceleration, k_y, is the acceleration that corresponds to a pseudo-static slope stability analysis for a critical failure surface with a seismic stability resistance ratio of $(D/C)_{EQ\text{-}Stability}$ = 1.0 (1/FS = 1.0).

The following are sources of uncertainty that are inherent when using the Newmark Time History method to compute displacements in the CEUS:

- Lack of strong motion time history records in the CEUS
- Earthquake source mechanism is not well understood
- Site-to-source distance, *R*, not well defined in the CEUS
- Infrequency of earthquake events in the last 10,000 years (Holocene Period)
- Point in the time history when cyclic strength degradation or liquefaction is triggered
- Magnitude of the apparent post-liquefaction residual resistance
- Influence of the thickness of liquefied soil on displacement
- Changes in values of yield acceleration, k_y, as deformation accumulates
- Influence of non-rigid sliding mass
- Influence of ground motion incoherence over the length of the sliding mass

Because of the uncertainties involved in the selection of the time history acceleration records in the CEUS, results of the Newmark Time History Analyses must be compared with the results obtained using Simplified Newmark Charts discussed in Section 13.17.2.

13.17.2 <u>Simplified Newmark Charts</u>

Simplified Newmark displacement charts were developed as a result of the NCHRP 12-70 (2007) study based on Newmark's Time History Analyses discussed in Section 13.17.1. These Simplified Newmark displacement charts are based on the earthquake database published by Hynes and Franklin (1984). The database of earthquake records used for this study was limited to earthquakes with moment magnitudes of $6.0 \leq M_w \leq 7.5$.

These charts have been developed as a function of a ratio (k_y / k_{max}) or yield acceleration (k_y) to max acceleration (k_{max}), peak ground velocity (V_{Peak} = PGV = V), peak ground acceleration, (PGA = A), and by region of the United States (WUS and CEUS) for either rock or soil site conditions. The charts shown in Figures 13-45 and 13-46 are based on an earthquake moment magnitude of $6.0 \leq M_w \leq 7.5$ in the CEUS. Figure 13-45 is appropriate for a stiff site with a peak ground velocity of V_{Peak} = 30 k_{max} in/sec (V_{Peak} = 760 k_{max} mm/sec). Figure 13-46 is appropriate for a soft soil site with a peak ground velocity of V_{Peak} = 60 k_{max} in/sec (V_{Peak} = 1520 k_{max} mm/sec).

For embankments and ERSs classified as ROC= I or II, the displacements computed using the simplified charts should be multiplied by 2 in order to obtain an 84% confidence level. For embankments and ERSs classified as ROC= III, do not multiply by any factor. The computed displacements should be compared with the required Performance Limits established in Chapter 10.

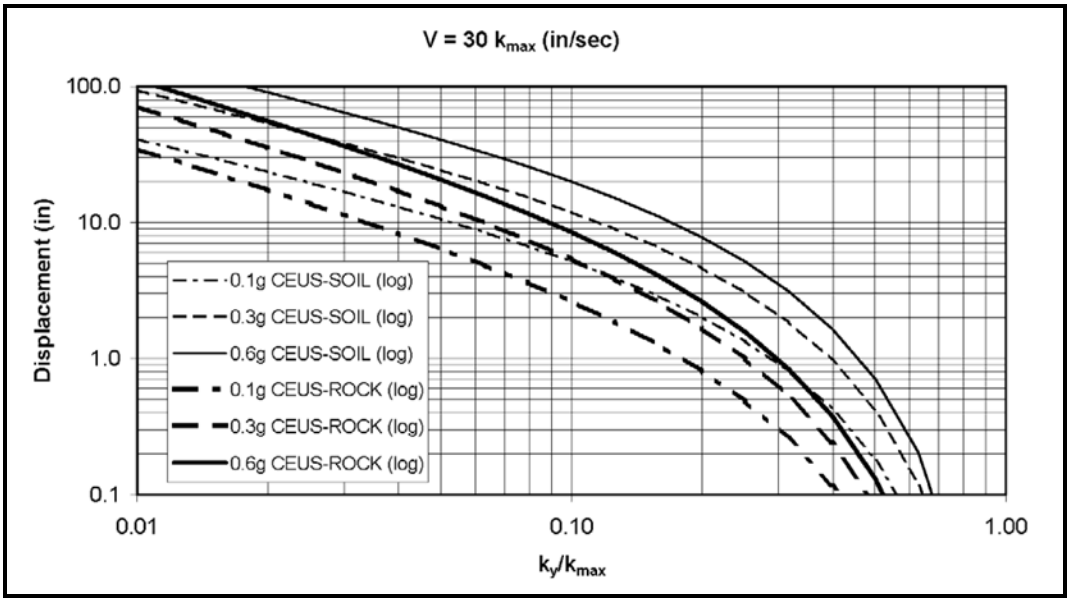


Figure 13-49, Simplified Newmark Chart (V = 30 k_{max} in/sec) (NCHRP 12-70, 2007)

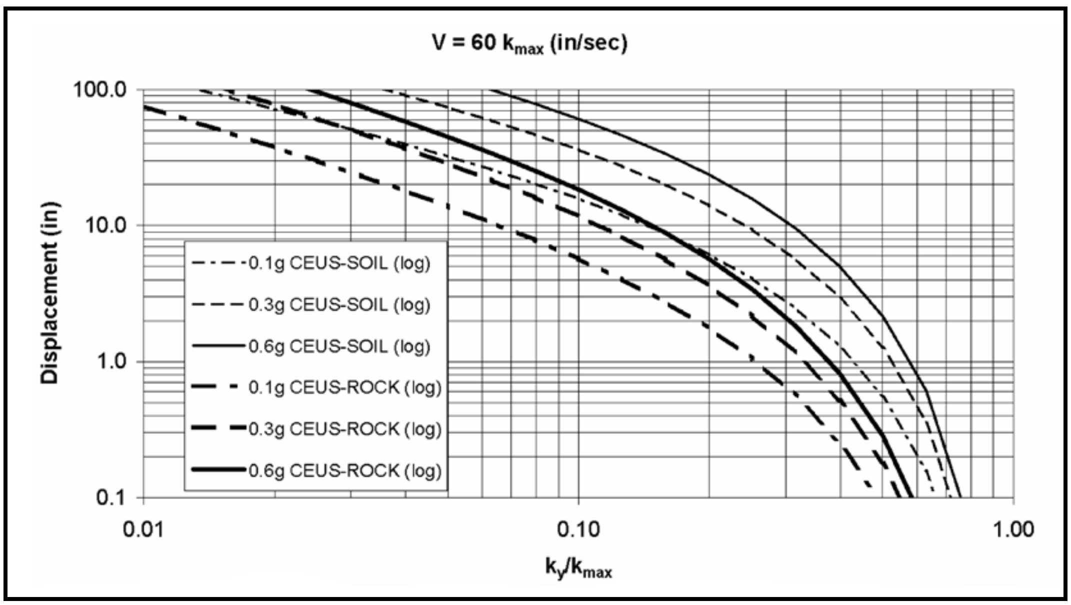


Figure 13-50, Simplified Newmark Chart (V = 60 k_{max} in/sec) (NCHRP 12-70, 2007)

In lieu of using charts in Figures 13-45 and 13-46 to compute the residual displacement, d, and having more flexibility in selecting the appropriate site factors (A and V_{Peak}), the following equations may be used.

CEUS-Rock (Standard Error of 0.31 $\log_{10}$ units):

Equation 13-105

$$\log(d) = -1.31 - 0.93\log\left(\frac{k_y}{k_{max}}\right) + 4.52\log\left[1-\left(\frac{k_y}{k_{max}}\right)\right] - 0.46\log\left(k_{max}\right) + 1.12\log\left(V_{Peak}\right)$$

CEUS-Soil (Standard Error of 0.23 $\log_{10}$ units):

Equation 13-106

$$\log(d) = -1.49 - 0.75\log\left(\frac{k_y}{k_{max}}\right) + 3.62\log\left[1-\left(\frac{k_y}{k_{max}}\right)\right] - 0.85\log(k_{max}) + 1.61\log(V_{Peak})$$

Where,

k_y = Yield Acceleration in units of g (Sections 13.15 and 13.17.1)

k_{max} = Peak ground acceleration (PGA) at the base of the failure surface in units of g

V_{Peak} = Peak ground velocity (PGV) in units of inches/sec. Correlations of peak ground velocity are found in Section 12.9.

13.18 SEISMIC SOIL SETTLEMENT

Ground settlements due to the effects of earthquake shaking (seismic induced) are one of the potential geotechnical seismic hazards that must be evaluated. Seismic induced ground settlements that are not due to flow failure or lateral spreading are typically the result of ground shaking caused densification of the underlying soils. Densification or seismic compression of soils has been observed in unsaturated sands, silts, and clayey sands above the water table. Densification of saturated loose sands subject to cyclic liquefaction has also been observed below the water table. Seismic settlements for depths greater than 80 feet, do not need to be computed unless the settlements are being computed to evaluate the effects of downdrag on deep foundations.

Soil settlements computed for unsaturated soils and saturated soils are additive as indicated by the following equation.

$$S_{TS} = S_{us} + S_{sat}$$ Equation 13-107

Where,

S_{TS} = Total seismic settlement in units of inches.

S_{us} = Total seismic settlement of unsaturated soils in units of inches (Section 13.18.3)

S_{sat} = Total seismic settlement of saturated soils in units of inches (Section 13.18.4)

The procedures presented in the following Sections for computing settlements are only applicable to level ground site conditions and in the absence of lateral flow or spreading. Soils susceptible to ground settlements that are located below sloping ground or adjacent to a free-face may be subject to static driving shear stresses oriented towards down-slope or free-face direction. The presence of static driving shear stresses for these site conditions will tend to increase vertical settlements and lateral displacements. Since the simplified methods presented to analyze settlements do not account for static driving shear stresses, the geotechnical engineer should be aware that settlements may be on the order of 10% to 20% greater for steeply sloped sites (Wu, 2002). This increase in settlement may be disregarded if the steeply sloped ground site has the potential for flow slide or seismic slope instability since these failure mechanisms will likely exceed the performance limits that have been established in Chapter 10.

13.18.1 Soil Characterization

The corrected SPT driving resistance ($N^*_{1,60}$) will be computed in accordance with Section 13.11.1.3. For soils with fines content (FC) greater than 5 percent, the SPT driving resistance must be adjusted for fines content to obtain an equivalent corrected clean sand SPT resistance ($N^*_{1,60,cs}$) in accordance with Section 13.11.2.

The normalized corrected CPT tip resistance ($q_{c,1,N}$) will be computed in accordance with Section 13.11.1.4. For soils with fines content (FC) greater than 5 percent, the normalized corrected CPT tip resistance must be adjusted for fines content to obtain an equivalent normalized corrected clean sand tip resistance ($q_{c,1,N,CS}$) in accordance with Section 13.11.3.

For seismic settlement methods that require an equivalent normalized corrected clean sand SPT resistance ($N^*_{1,60,cs}$) and only CPT in-situ testing data is available to compute seismic settlements, the normalized cone tip resistance, $q_{c,1,N}$, (Section 13.11.1.4) needs to be correlated to corrected SPT $N^*_{1,60}$ values in accordance with Section 13.11.1.1. The correlated SPT blow count ($N^*_{1,60}$) should then be adjusted for fines content (FC) greater than 5 percent, in accordance with Section 13.11.2.

13.18.2 Cyclic Shear Strain (γ)

The cyclic shear strain, γ, induced during earthquake shaking at various depths below the ground surface is the primary factor that controls the magnitude of the settlement of unsaturated sands. The effective shear strain, γ_{eff}, (or average shear strain, γ) is used to estimate seismic settlement by using the following equation.

$$\gamma_{eff} = \frac{\tau_{avg}}{G_{eff}} = \frac{\tau_{avg}}{G_{\max}\left(\frac{G_{eff}}{G_{\max}}\right)} \qquad \text{Equation 13-108}$$

Where,

- τ_{avg} = Average cyclic shear stress
- G_{eff} = Effective shear modulus at the induced strain level. The ratio of the effective shear modulus (G_{eff}) to initial small strain shear modulus (G_{max}) is a hypothetical effective shear stress factor (G_{eff}/G_{max}). The effective shear strain, γ_{eff}, can be computed as a function of γ_{eff} (G_{eff}/G_{max}) using the following equation.

$$\gamma_{eff}\left(\frac{G_{eff}}{G_{\max}}\right) = \frac{\tau_{avg}}{G_{\max}} \qquad \text{Equation 13-109}$$

The *Simplified Procedure* (Section 13.10.1.1) for determining the maximum earthquake induced stress (τ_{max}) should be used for the upper 80 feet. For depths greater than 80 feet the maximum earthquake induced stress (τ_{max}) should be computed using site specific response analyses in accordance with Section 13.10.1.2.

The average cyclic shear stress, τ_{avg}, is computed by taking 65% of the maximum earthquake induced stress (τ_{max}) as shown in the following equation.

$$\tau_{avg} = 0.65\tau_{max} \quad \text{Equation 13-110}$$

The initial small strain shear modulus, G_{max}, or soil stiffness can be determined based on the mass density, ρ, of the soil and the shear wave velocity, V_s, of the soil as indicated in Chapter 12.

The γ_{eff} (G_{eff}/G_{max}) variable is then computed using Equation 13-102. The effective shear strain, ($\gamma_{eff} = \gamma$) can be estimated using the relationships developed by Darendeli and Stokoe (2001), Vucetic and Dobry (1991), and Iwasaki et al. (1978) for modulus reduction curves shown in Figure 13-47.

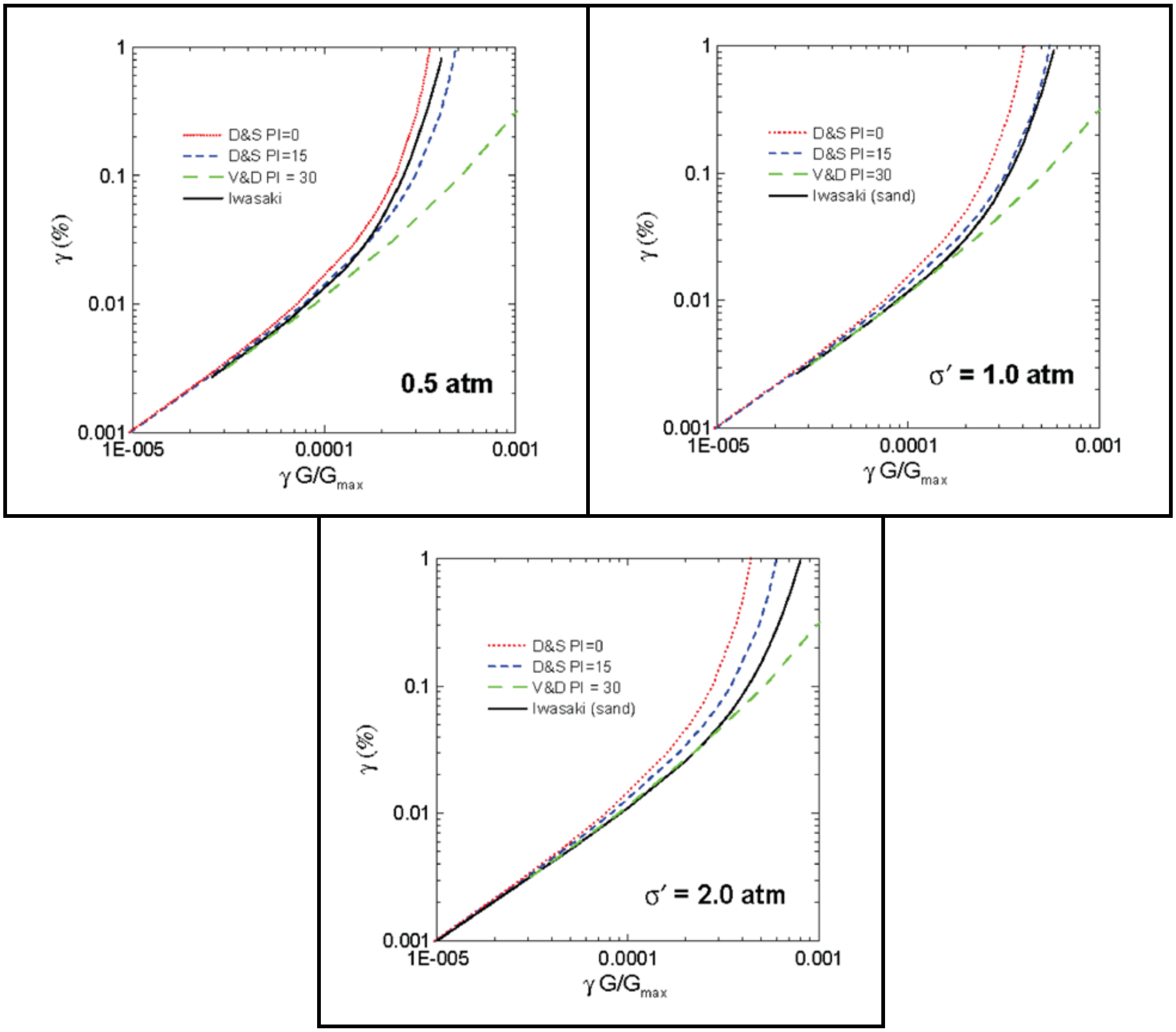


Figure 13-51, Modulus Reduction Curves (presented by Stewart et al., 2004)

The effective shear strain ($\gamma_{eff} = \gamma$) relationships presented in Figure 13-47 for a plasticity index, PI = 0, 15, and 30 and effective overburden stress, σ'_v = 0.5atm, 1.0 atm, 2.0 atm can be computed by the using the following equation. For soils with (0 < PI < 15) or (15 < PI < 30) the shear strains (γ) can be linearly interpolated using the following equation.

$$\gamma = \frac{\left(1 + \left(g_1 e^{(g_2 \cdot P)}\right)\right)}{1 + g_1} P \cdot 100 \qquad \text{Equation 13-111}$$

Where,

PI ≈ 0:

$g_1 = 0.199(\sigma'_m/p_a)^{0.231}$

$g_2 = 10850(\sigma'_m/p_a)^{-0.410}$

PI ≈ 15:

$g_1 = 0.194(\sigma'_m/p_a)^{0.265}$

$g_2 = 7490(\sigma'_m/p_a)^{-0.418}$

PI ≈ 30:

$g_1 = 4.0$

$g_2 = 1400$

p_a = Atmospheric pressure: pa = 1.0 atm = 101.3 kPa = 1.0 tsf

σ'_m = Mean effective stress. An approximation of mean normal effective stress, $\sigma'_m \approx 0.65\sigma'_v$ would apply to a sand layer with an approximate internal friction angle, ϕ, of 32 degrees. σ'_m can be computed by the following equation:

$$\sigma'_m = \left[\frac{1+2K_o}{3}\right]\sigma'_v \approx 0.65\sigma'_v \qquad \text{Equation 13-112}$$

K_o = Coefficient of lateral earth pressure at-rest (Chapter 12)

σ'_v = Effective vertical overburden stress. In units of tons per square foot (tsf)

p_a = Atmospheric pressure

$P = \gamma_{eff}\left(\frac{G_{eff}}{G_{max}}\right)$ (See Equation 13-102)

13.18.3 Unsaturated (Dry) Sand Settlement

Settlement of unsaturated loose sand (above the water table) typically occurs during the earthquake shaking under conditions of constant effective vertical stress (depending on the degree of saturation). Sands that are above the water table are typically referred to as unsaturated, even though these soils typically have some degree of saturation particularly near the water table. Settlement potential of unsaturated sands is computed using the same methodology given by Tokimatsu and Seed (1987) except that volumetric strain models developed by Stewart et al. (2004) are used. The total unsaturated soil settlement, S_{us}, is computed as the sum of the individual settlement of the unsaturated soil settlement layers, δ_{us}, as indicated in the following equation.

$$\boldsymbol{S_{us}} = \mathbf{2}\cdot\sum_{n=1}^{n=i_{us}}\delta_{us} = \mathbf{2}\cdot\sum_{n=1}^{n=i_{us}}\varepsilon_c \boldsymbol{C_{VN}}\cdot\boldsymbol{H_{us}} \qquad \text{Equation 13-113}$$

Where,

δ_{us} = Settlement of Unsaturated Sand layer in units of inches

ε_c = Volumetric Strain of the in-place soil (or compacted fill) in units of (%)

C_{VN} = Volumetric strain correction factor

H_{us} = Layer Thickness of Unsaturated Sand layer in units of inches

i_{us} = total number (n) of unsaturated sand layers

Note that a factor of two is included in Equation 13-106 to correct for multidirectional shaking effect as recommended by Pyke et al. (1975).

13.18.3.1 Volumetric Strain Soil Models

The cyclic shear strain, γ_c, at each soil layer can be approximated by the effective shear strain, $\gamma_{eff,}$ ($\gamma_c \approx \gamma_{eff} = \gamma$) as computed in Section 13.18.2. The volumetric strain of the unsaturated soil, ε_c, for an earthquake of equivalent number of uniform strain cycles of n = 15 can be determined from volumetric strain material models based on laboratory simple shear testing. Volumetric strain material models are provided for clean sand, soil with non-plastic fines, and soils with variable plastic fines in Sections 13.18.3.3, 13.18.3.4, and 13.18.3.5, respectively.

The volumetric soil models provided for clean sands uses relative density, D_r = 45%, 60%, and 80%, as an indicator for selecting model coefficients. For in place soils, the relative density can be estimated based on the in-situ testing correlations provided in Section 13.11.1.1. For compacted fill embankments, a relative density, D_r = 60% can be used.

The volumetric soil models for soils with non-plastic fines and variable plastic fines use a relative compaction (RC) based on the Modified Proctor test and the degree of saturation parameter, S. Since the SCDOT standard construction specifications use the Standard Proctor test for field control, it is necessary to estimate the equivalent RC based on the Modified Proctor test for the various soil models. The degree of saturation, S, for in-place soils shall be computed from laboratory testing. The degree of saturation, S, for compacted soils will need to be estimated. Table 13-12 provides approximate relationships between the Standard and Modified Proctor value and approximate degree of saturation, S, for the maximum dry density of various soil types. The values in Table 13-12 are estimates based on accepted industry practice and shall be used unless specific testing has been performed. The Proctor test values shall be determined using the testing methods provided in Chapter 5.

Table 13-12, Relationships for Relative Compaction and Saturation

Soil Types	Description	Standard Proctor	Modified Proctor [1]	Degree of Saturation
Soils With NP Fines	50 % Sand / 50% Silt	95%	85%	55%
		100%	90%	66%
Soils with Variable Plasticity Fines	Low Plasticity Fine Grained Soil (PI = 2)	95%	85%	58%
		100%	90%	66%
	Moderate Plasticity Fine Grained Soil (PI = 15)	95%	78%	67%
		100%	82%	75%
	High Plasticity Fine Grained Soil (PI = 27)	95%	76%	75%
		100%	80%	83%

[1] The equivalent Standard Proctor RC is based on the following:
90% of the Standard Proctor for Silty Soils
82% of the Standard Proctor for Moderately Plastic Clay Soils
80% of the Standard Proctor for Highly Plastic Clay Soils

The ratio between the volumetric strain of the in-place soils (ε_c) and the equivalent volumetric strain (ε_{c-15}) of the soil for a number of uniform strain cycles, N = 15 is defined as the volumetric strain correction factor. The equivalent number of uniform strain cycles (N) can be estimated from Section 13.18.3.2. The volumetric correction ratio is provided for each volumetric strain material model in the following Sections.

13.18.3.2 Equivalent Number of Uniform Strain Cycles (N)

The equivalent number of uniform strain cycles (N) is used to normalize the effects of earthquakes of different Moment Magnitudes (M_W = m) and site-to-source distances (R = r). The relationship shown in Figure 13-48 was developed by Liu et al. (2001) to estimate the equivalent number of uniform strain cycles (N). Also shown in Figure 13-48 is Seed et al. (1975) recommendations.

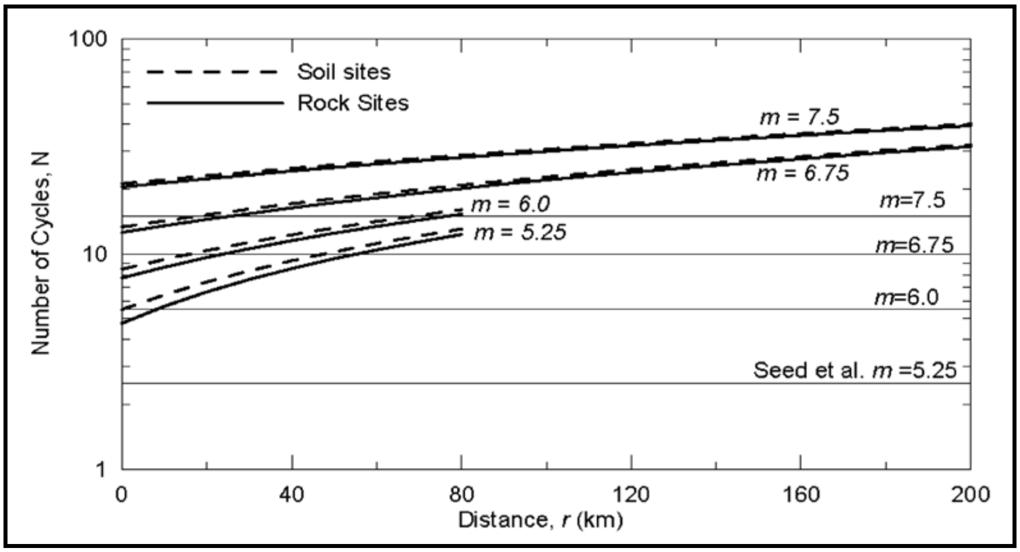


Figure 13-52, Number of Cycles with Distance and Moment Magnitude (Liu et al., 2001 and Seed et al., 1975 with permission from ASCE)

The Liu et al. (2001) relationship for the equivalent number of uniform strain cycles (N) can be computed using the following equation.

$$\boldsymbol{N} = \frac{\left(\exp\left(\boldsymbol{b_1} + \boldsymbol{b_2}\left(\boldsymbol{M_W} - \boldsymbol{m^*}\right)\right)\right)^{\left(\frac{1}{3}\right)}}{\mathbf{4.9} \times \mathbf{10^6}\,\boldsymbol{\beta}} + \boldsymbol{Sc_1} + \boldsymbol{Rc_2} \qquad \text{Equation 13-114}$$

Where,

b_1	=	1.53 (Coefficient)
b_2	=	1.51 (Coefficient)
c_1	=	0.750 (Coefficient)
c_2	=	0.095 (Coefficient)
β	=	*3.2* (Coefficient)
M_W	=	Earthquake moment magnitude
m^*	=	5.8

S = Site Parameter: $S = 0$ (Rock Site or Soil < 20 m) or $S = 1$ (Soil Site > 20 m)

R = Site-Source Distance in units of kilometers (km)

13.18.3.3 Clean Sand

Clean sand is typically defined as a cohesionless material with fines content, FC, less than 5 percent. The volumetric strain model for Clean Sand proposed by Stewart et al. (2004) with relative density (D_r) of 45%, 60%, and 80% is shown in Figure 13-49.

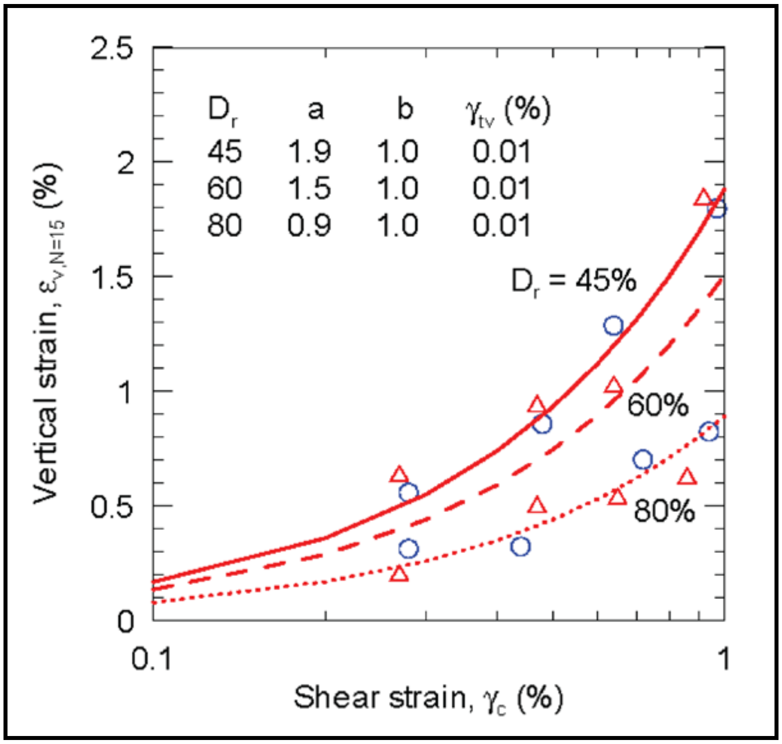


Figure 13-53, Volumetric Strain Model – Clean Sand (Stewart et al., 2004)

The volumetric strain model for Clean Sand proposed by Stewart et al. (2004) can be computed by the following equations. Note that these correlations are applicable to cyclic strains less than 1.0 percent ($\gamma_c < 1.0\%$).

$$\gamma_c > \gamma_{tv}: \quad \varepsilon_{v,N=15} = a(\gamma_c - \gamma_{tv})^b \qquad \text{Equation 13-115}$$

$$\gamma_c < \gamma_{tv}: \quad \varepsilon_{v,N=15} = 0 \qquad \text{Equation 13-116}$$

Where,

$\varepsilon_{v,N=15}$ = Volumetric strain of unsaturated soils in units of percentage (%)

$\gamma = \gamma_c$ = Cyclic strain in units of percentage (%) (Equation 13-104)

γ_{tv} = Thresholds shear strain in units of percentage (%) – See Table 13-13

a = Coefficient – See Table 13-13

b = Coefficient – See Table 13-13

Table 13-13, Volumetric Strain Clean Sand Model Coefficients

D_r (%)	a	b	γ_{tv} (%)
45	1.9	1.0	0.01
60	1.5	1.0	0.01
80	0.9	1.0	0.01

The volumetric strain correction factor, C_{VN}, adjusts the volumetric strain for N=15 cycles, $\varepsilon_{v,N=15}$, obtained from the volumetric soil model to the equivalent number of uniform strain cycles (*N*) of the design earthquake (FEE or SEE) being evaluated. The equivalent number of uniform strain cycles (N) is computed using the moment magnitude (M_W = m) and site-to-source distances (R) as indicated in Section 13.18.3.2. The volumetric strain correction factor, C_{VN}, for Clean Sands is computed using the following equation.

$$\boldsymbol{C_{VN}} = \mathbf{0.33}\ln(\boldsymbol{N}) + \mathbf{0.1063} \qquad \text{Equation 13-117}$$

13.18.3.4 Soils With Non-Plastic Fines

Soils with non-plastic fines are defined as a cohesionless material with 50% Sand / 50% Silt by weight. The volumetric strain model for soils with non-plastic fines proposed by Stewart et al. (2004) with Relative Compaction (RC) based on the Modified Proctor of RC= 87% and 92% with corresponding degree of saturation of S = 30% and 0, ≥60% for each RC are shown in Figure 13-50.

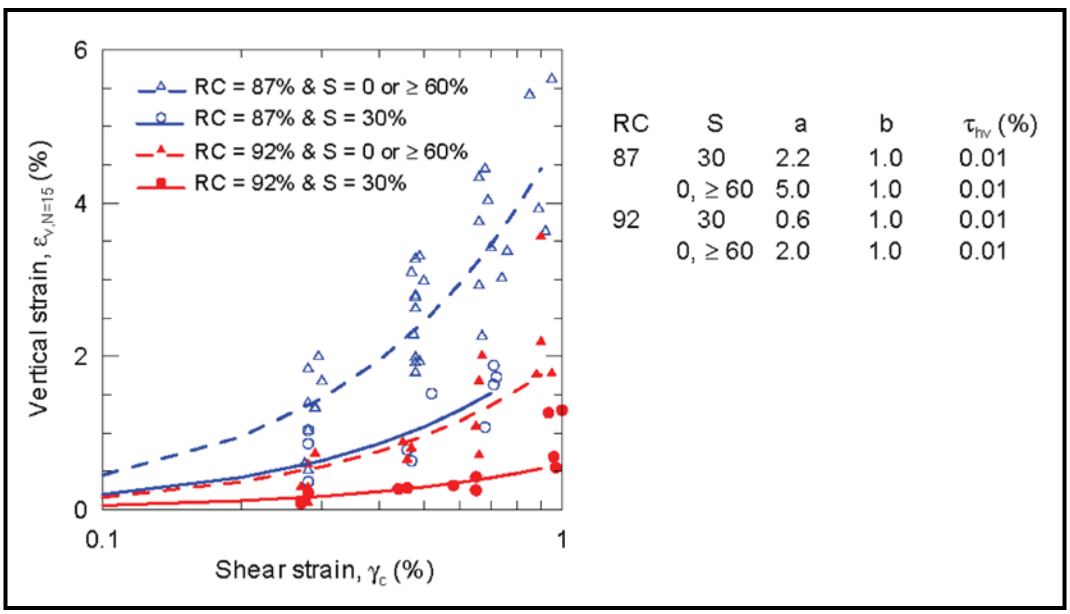


Figure 13-54, Volumetric Strain Model - Soils With Non-plastic Fines (Stewart et al., 2004)

The volumetric strain model for soils with non-plastic fines proposed by Stewart et al. (2004) can be computed by the following equations. Note that these correlations are applicable to cyclic strains less than 1.0 percent (γ_c < 1.0%).

$$\gamma_c > \gamma_{tv}: \quad \varepsilon_{v,N=15} = a(\gamma_c - \gamma_{tv})^b \qquad \text{Equation 13-118}$$

$$\gamma_c < \gamma_{tv}: \quad \varepsilon_{v,N=15} = 0 \qquad \text{Equation 13-119}$$

Where,

$\varepsilon_{v,N=15}$ = Volumetric strain of unsaturated soils in units of percentage (%)

$\gamma = \gamma_c$ = Cyclic strain in units of percentage (%) (Equation 13-104)

γ_{tv} = Thresholds shear strain in units of percentage (%) – See Table 13-14

a = Coefficient – See Table 13-14

b = Coefficient – See Table 13-14

Table 13-14, Volumetric Strain Soils With Non-Plastic Fines Model Coefficients

RC (%)	S (%)	a	b	γ_{tv} (%)
87	30	2.2	1.0	0.01
87	0, >60	5.0	1.0	0.01
92	30	0.60	1.0	0.01
92	0, >60	2.0	1.0	0.01

For soils with fines content (Silt) between 0 and 50%, the volumetric strain for these types of soils should be interpolated between the results of this model (soils with non-plastic fines) and the results of the volumetric strain model for clean sand (Section 13.18.3.3).

The volumetric strain correction factor, C_{VN}, adjusts the volumetric strain for N=15 cycles, $\varepsilon_{v,N=15}$, obtained from the volumetric soil model to the equivalent number of uniform strain cycles (N) of the design earthquake (FEE or SEE) being evaluated. The equivalent number of uniform strain cycles (N) is computed using the moment magnitude (M_W = m) and site-to-source distances (R) as indicated in Section 13.18.3.2. The volumetric strain correction factor, C_{VN}, for soils with non-plastic fines is computed using the following equation.

$$C_{VN} = 0.36 \ln(N) + 0.0251 \qquad \text{Equation 13-120}$$

13.18.3.5 Soils With Variable Plastic Fines

Soil with variable plastic fines is defined as a fine-grained soil with fine contents, FC > 50% and a range of Plasticity Index of 2 ≤ PI ≤ 27. These soils have been divided into Low Plasticity Soil, Moderate Plasticity Soil, and High Plasticity Soil as indicated below.

Low Plasticity Soils: A volumetric strain model for Low Plasticity Soils (2 ≤ PI ≤ 10, LL=27) was proposed by Stewart et al. (2004) with Relative Compaction (RC) based on the Modified Proctor of RC = 90-92% and 93-95% and variation in degree of saturation, S=55-80 and S=80-99% for each RC range as are shown in Figure 13-51. The overall strain level for Low Plasticity Soils was significantly less than those for clean sand and soils with non-plastic fines (Silts).

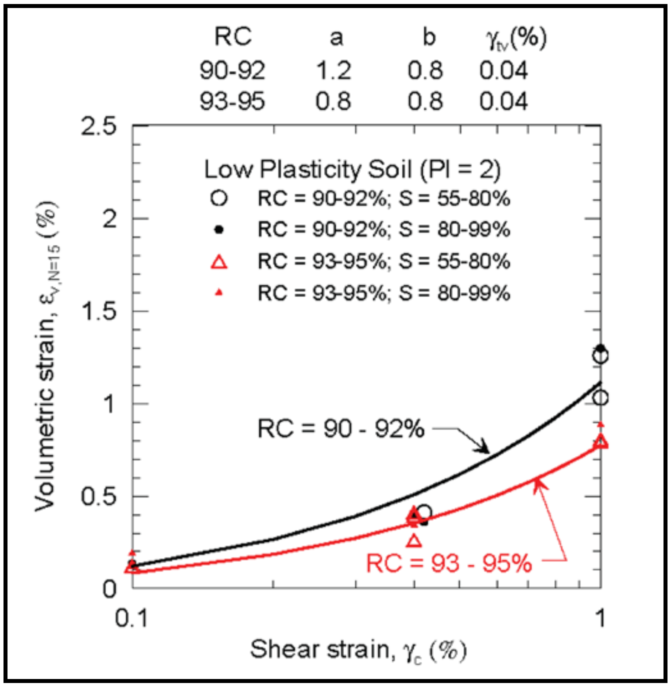


Figure 13-55, Volumetric Strain Model – Low Plasticity Soil (Stewart et al., 2004)

The volumetric strain model for Low Plasticity Soil proposed by Stewart et al. (2004) can be computed by the following equations. Note that these correlations are applicable to cyclic strains less than 1.0 percent (γ_c < 1.0%).

$$\gamma_c > \gamma_{tv}: \quad \varepsilon_{v,N=15} = a(\gamma_c - \gamma_{tv})^b \qquad \text{Equation 13-121}$$

$$\gamma_c < \gamma_{tv}: \quad \varepsilon_{v,N=15} = 0 \qquad \text{Equation 13-122}$$

Where,

$\varepsilon_{v,N=15}$ = Volumetric strain of unsaturated soils in units of percentage (%)

$\gamma = \gamma_c$ = Cyclic strain in units of percentage (%) (Equation 13-104)

γ_{tv} = Thresholds shear strain in units of percentage (%) – See Table 13-15

a = Coefficient – See Table 13-15

b = Coefficient – See Table 13-15

Table 13-15, Volumetric Strain Low Plasticity Soil Model Coefficients

RC (%)	a	b	γ_{tv} (%)
90 - 92	1.2	0.8	0.04
93 - 95	0.8	0.8	0.04

The volumetric strain correction factor, C_{VN}, adjusts the volumetric strain for N=15 cycles, $\varepsilon_{v,N=15}$, obtained from the volumetric soil model to the equivalent number of uniform strain cycles (N) of the design earthquake (FEE or SEE) being evaluated. The equivalent number of uniform strain cycles (N) is computed using the moment magnitude (M_W = m) and site-to-source distances (R) as indicated in Section 13.18.3.2. The volumetric strain correction factor, C_{VN}, for Low Plasticity Soil is computed using the following equation.

$$C_{VN} = 0.32\ln(N) + 0.1334 \qquad \text{Equation 13-123}$$

Moderate Plasticity Soils: A volumetric strain model for Moderate Plasticity Soil ($11 \leq PI \leq 20$, LL=33) was proposed by Stewart et al. (2004) with Relative Compaction (RC) based on the Modified Proctor of RC= 84%, 88%, 88%, 92%, and 92% with corresponding degree of saturation, S = Any, 60%, 90%, 70%, and 90% was developed as are shown in Figure 13-52. It was observed that volumetric strain decreases with increasing RC and S.

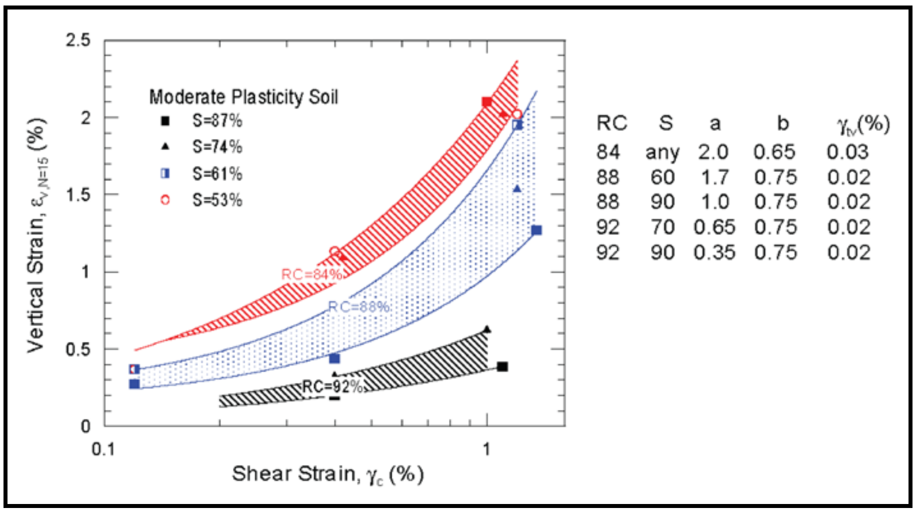


Figure 13-56, Volumetric Strain Model – Moderate Plasticity Soil (Stewart et al., 2004)

The volumetric strain model for Moderate Plasticity Soil proposed by Stewart et al. (2004) can be computed by the following equations. Note that these correlations are applicable to cyclic strains less than 1.0 percent ($\gamma_c < 1.0\%$).

$$\gamma_c > \gamma_{tv}: \quad \varepsilon_{v,N=15} = a(\gamma_c - \gamma_{tv})^b \qquad \text{Equation 13-124}$$

$$\gamma_c < \gamma_{tv}: \quad \varepsilon_{v,N=15} = 0 \qquad \text{Equation 13-125}$$

Where,

$\varepsilon_{v,N=15}$	=	Volumetric strain of unsaturated soils in units of percentage (%)
$\gamma = \gamma_c$	=	Cyclic strain in units of percentage (%) (Equation 13-104)
γ_{tv}	=	Thresholds shear strain in units of percentage (%) – See Table 13-16
a	=	Coefficient – See Table 13-16
b	=	Coefficient – See Table 13-16

Table 13-16, Volumetric Strain Moderate Plasticity Soil Model Coefficients

RC (%)	S (%)	a	b	γ_{tv} (%)
84	Any	2.00	0.65	0.03
88	60	1.70	0.75	0.02
88	90	1.00	0.75	0.02
92	70	0.65	0.75	0.02
92	90	0.35	0.75	0.02

The volumetric strain correction factor, C_{VN}, adjusts the volumetric strain for N=15 cycles, $\varepsilon_{v,N=15}$, obtained from the volumetric soil model to the equivalent number of uniform strain cycles (*N*) of the design earthquake (FEE or SEE) being evaluated. The equivalent number of uniform strain cycles (N) is computed using the moment magnitude (M_W = m) and site-to-source distances (R) as indicated in Section 13.18.3.2. The volumetric strain correction factor, C_{VN}, for Moderate Plasticity Soil is computed using the following equation.

$$C_{VN} = 0.34\ln(N) + 0.0793 \qquad \text{Equation 13-126}$$

High PlasticitySoils: A volumetric strain model for High Plasticity Soil (21 ≤ PI ≤ 27, LL=47) was proposed by Stewart et al. (2004) with Relative Compaction (RC) based on the Modified Proctor of RC= 87% with corresponding degree of saturation, S = 60%, 90%, and RC= 92% any saturation, was developed as are shown in Figure 13-53. It was observed that volumetric strain showed dependency on saturation at low RC = 87% and relatively no dependency on saturation high at RC =92%. These soils resulted in volumetric strains that were about half of the Moderate Plastic soils. For soils with PIs greater than 27, no volumetric strain is anticipated.

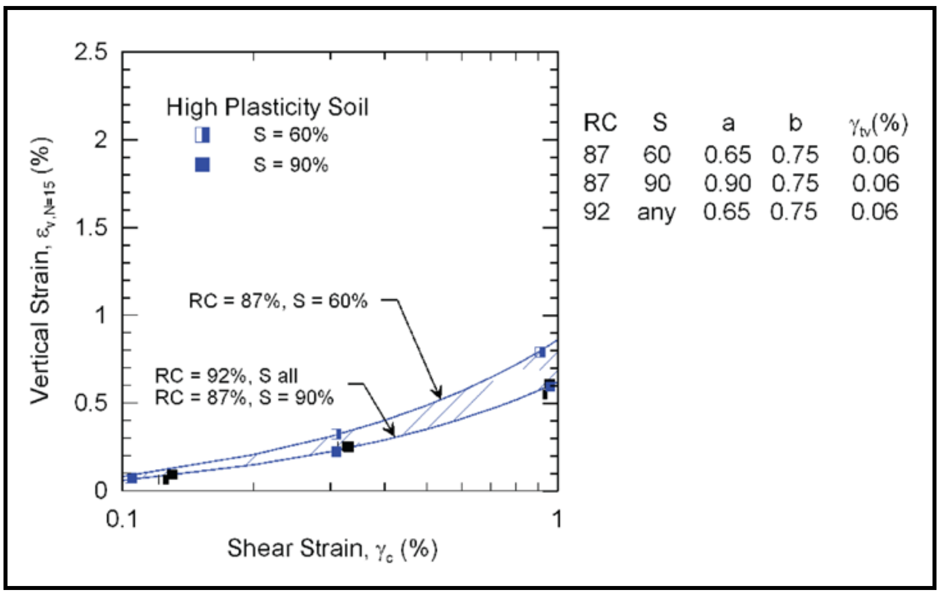


Figure 13-57, Volumetric Strain Model – High Plasticity Soil (Stewart et al., 2004)

The volumetric strain model for High Plasticity Soils proposed by Stewart et al. (2004) can be computed by the following equations. Note that these correlations are applicable to cyclic strains less than 1.0 percent (γ_c < 1.0%).

$$\gamma_c > \gamma_{tv}: \quad \varepsilon_{v,N=15} = a(\gamma_c - \gamma_{tv})^b \qquad \text{Equation 13-127}$$

$$\gamma_c < \gamma_{tv}: \quad \varepsilon_{v,N=15} = 0 \qquad \text{Equation 13-128}$$

Where,

$\varepsilon_{v,N=15}$ = Volumetric strain of unsaturated soils in units of percentage (%)
$\gamma = \gamma_c$ = Cyclic strain in units of percentage (%) (Equation 13-104)
γ_{tv} = Thresholds shear strain in units of percentage (%) – See Table 13-17
a = Coefficient – See Table 13-17
b = Coefficient – See Table 13-17

Table 13-17, Volumetric Strain High Plasticity Soil Model Coefficients

RC (%)	S (%)	a	b	γ_{tv} (%)
87	60	0.65	0.75	0.06
87	90	0.90	0.75	0.06
92	Any	0.65	0.75	0.06

The volumetric strain correction factor, C_{VN}, adjusts the volumetric strain for N=15 cycles, $\varepsilon_{v,N=15}$, obtained from the volumetric soil model to the equivalent number of uniform strain cycles (N) of the design earthquake (FEE or SEE) being evaluated. The equivalent number of uniform strain cycles (N) is computed using the moment magnitude (M_W = m) and site-to-source distances (R) as indicated in Section 13.18.3.2. The volumetric strain correction factor, C_{VN}, for High Plasticity Soil is computed using the following equation.

$$C_{VN} = 0.25\ln(N) + 0.3230 \qquad \text{Equation 13-129}$$

13.18.4 Saturated Sand Settlement

Settlement of saturated sands occurs when Sand-Like soils have the potential to experience cyclic liquefaction due to dissipation of excess pore water pressure generated during the earthquake shaking. These soils experience a reconsolidation as the pore water pressure dissipates and the sand particles rearrange into a more compact state causing settlement of these soil layers. Seismic induced settlements of Sand-Like soils that have the potential to experience cyclic liquefaction are typically larger than compaction settlements that result from unsaturated sands. Several methods to evaluate the magnitude of seismic settlement of saturated sands have been proposed by Tokimatsu and Seed (1987), Ishihara and Yoshimine (1992), Shamoto et al. (1998), and Wu et al. (2003). These methods all use the reconsolidation volumetric strain due to cyclic liquefaction (ε_v). The total settlement, S_{sat}, of Sand-Like Soils with the potential to experience cyclic liquefaction is computed using the following equation.

$$S_{sat} = \sum_{n=1}^{n=i_{sat}} \delta_{sat} = \sum_{n=1}^{n=i_{sat}} \varepsilon_v \cdot H_{sat} \qquad \text{Equation 13-130}$$

Where,

δ_{sat} = Post-Liquefaction Settlement of Saturated Sand layer in units of inches

ε_v = Reconsolidation Volumetric Strain due to Liquefaction in units of percentage (%)

H_{sat} = Layer Thickness of Saturated Sand layer in units of inches

i_{sat} = Total number (n) of Potentially Liquefiable sand layers

The most referenced method used to compute the settlement potential of saturated liquefiable clean sands was proposed by Tokimatsu and Seed (1987). Several other relationships (Ishihara and Yoshimine, 1992; Shamoto et al, 1998 and Wu et al., 2002) have been proposed that address some of the deficiencies found with Tokimatsu and Seed, (1987) with respect to soils that have higher fines content. Idriss and Boulanger (2008) compared these three alternate methods as shown in Figure 13-54.

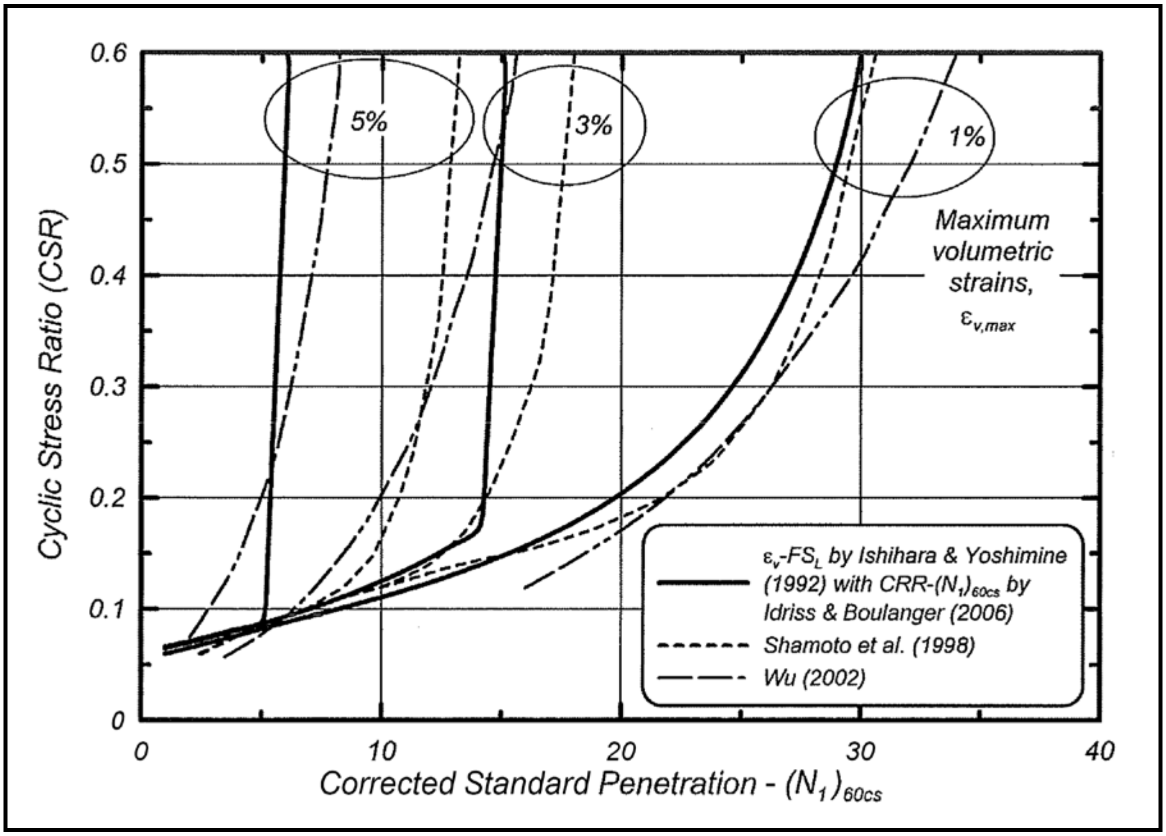
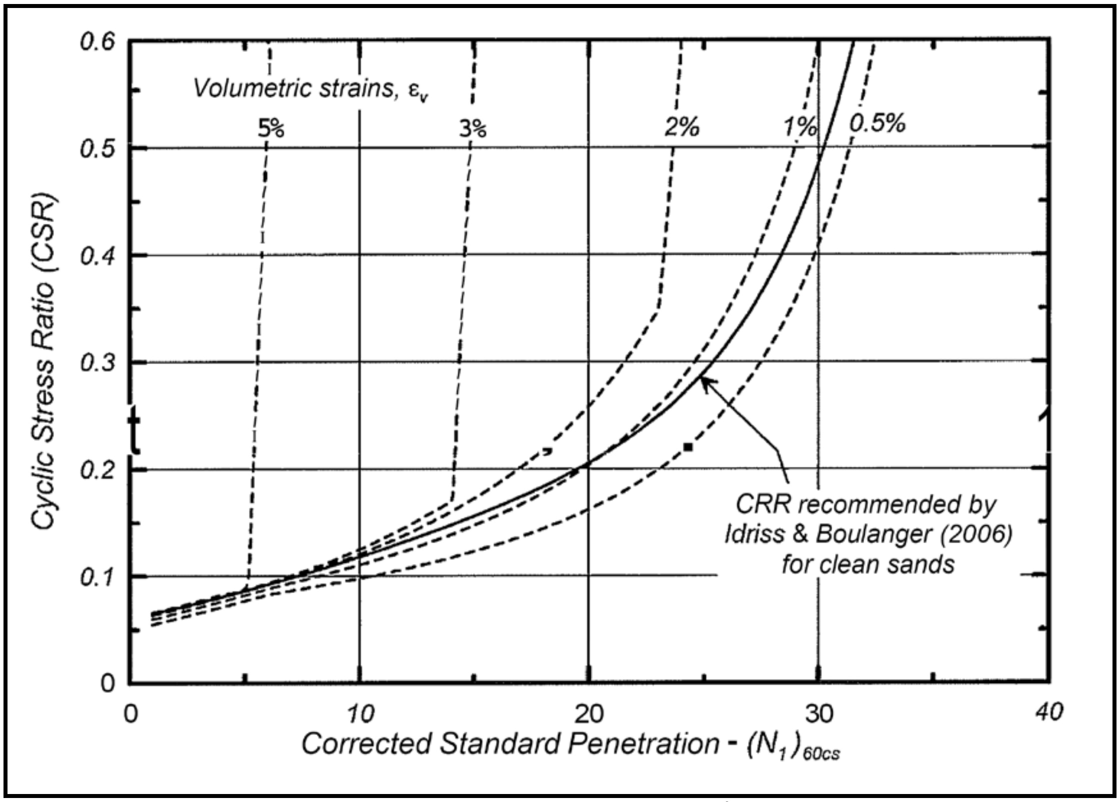


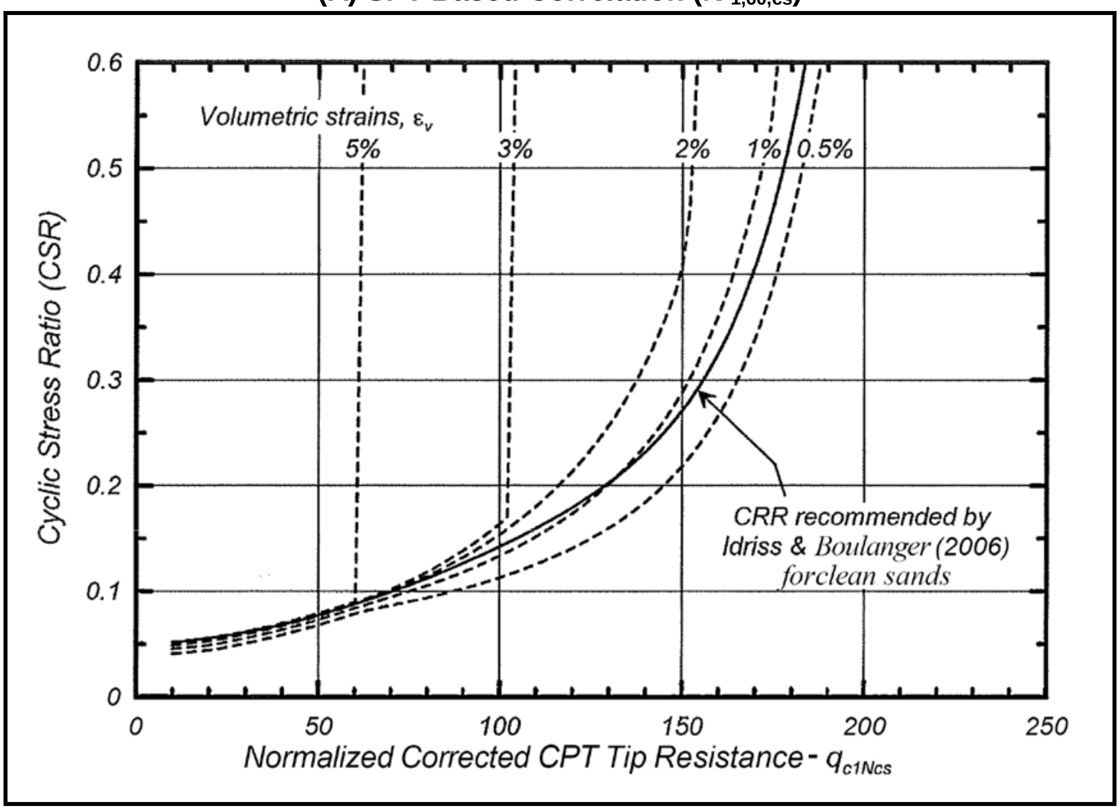
Figure 13-58, Volumetric Strain Relationship Comparison - M_w=7.5; σ'_{vc} = 1 atm (Idriss and Boulanger, 2008)

The settlement of saturated sands that are potentially liquefiable will be computed based on Idriss and Boulanger (2008) recommended reconsolidation volumetric strain, ε_v, relationship based on Ishihara and Yoshimine (1992) shown in Figure 13-55. Reconsolidation volumetric strain, ε_v, relationships for SPT and CPT results in Figure 13-55 have been developed to be compatible by using the correlations for relative density from SPT and CPT in Section 13.11.1.1. The CSR^*_{eq}, is computed based on Section 13.10. The normalized SPT driving resistance, $N^*_{1,60,cs}$, is computed based on Section 13.11.2. The normalized corrected CPT tip resistance, $q_{c,1,N,CS}$, is computed based on Section 13.11.3.

The use of reconsolidation volumetric strain, ε_v, relationship based on Shamoto et al. (1998), or Wu et al. (20032) will require approval from the PCS/GDS. If CPT testing data is used with these relationships, the correlations for relative density from SPT and CPT in Section 13.11.1.1 shall be used in order to maintain compatibility between testing methods.



(A) SPT Based Correlation ($N^*_{1,60,cs}$)



(B) CPT Based Correlation ($q_{c,1,N,cs}$)

Figure 13-59, Volumetric Strain Relationship - M_w=7.5; σ'_{vc} = 1 atm (Ishihara and Yoshimine, 1992; modified by Idriss and Boulanger, 2008)

When soils are stratified and potentially cyclic liquefiable layers are located between non-liquefiable soil layers, there is a possibility of under-predicting or over-predicting excess pore water developed depending on the location of the soil layers within the stratified system (Polito and Martin, 2001). Polito and Martin (2001) have shown that thin layers of dense sand (non-liquefiable soil) could liquefy if sandwiched between liquefiable soil layers. Ishihara (1985) proposed the method shown in Figure 13-56 to determine the thickness, H_2, of the liquefiable soil layer. H_1 is the thickness of the non-liquefiable soil layer above the liquefiable soil layer, H_2. The thickness of the liquefiable soil layer, H_2, is dependent on criteria indicated in Figure 13-56. In addition to the criteria indicated in Figure 13-56, the following criteria must also be satisfied:

1. Thickness of the non-liquefiable layer (H_b) is less than or equal to 5 feet ($H_b \leq 5$ feet).

2. Non-liquefiable soil layer "B" has a normalized and corrected SPT $N^*_{1,60,cs} < 30$ blows/foot or a normalized corrected CPT tip resistance $q_{1,c,N,cs} < 170$.

3. Non-liquefiable soil layer "B" is a sand or silty sand with fines content, $FC \leq 35$.

4. Moment magnitude of design earthquake, $M_w \geq 7.0$.

This procedure to evaluate thickness, H_2, of liquefiable soil layers is used for all subsequent soil layers that have the potential to liquefy in the stratified soil system.

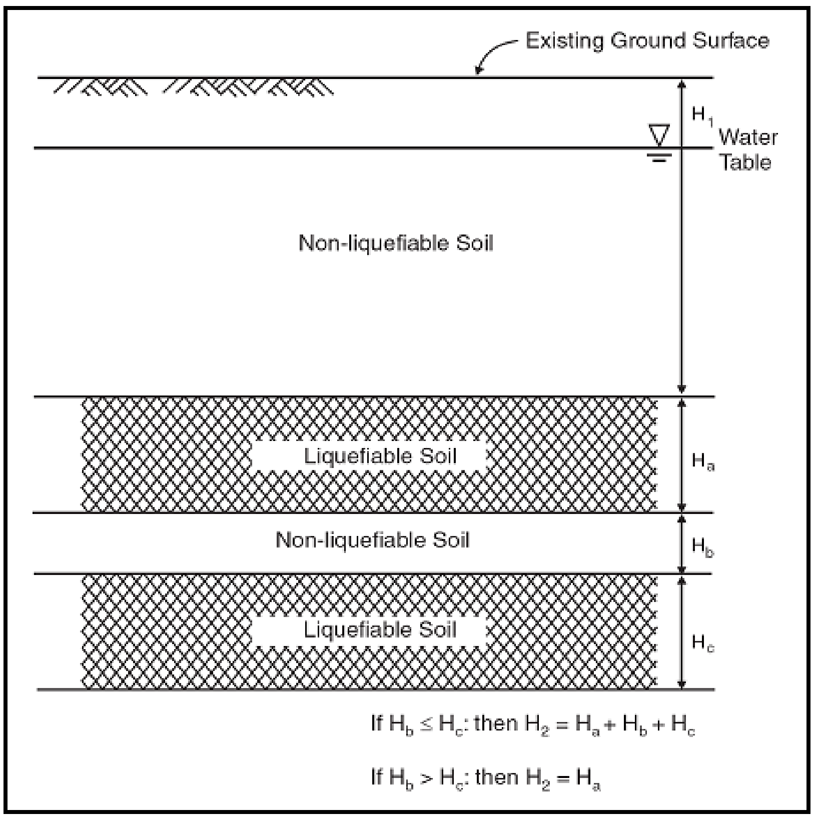


Figure 13-60, Liquefiable Soil Layer Thickness in Stratified Soils (Ishihara, 1985)

13.19 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.

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Chapter 14

GEOTECHNICAL SEISMIC DESIGN

FINAL

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

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CHAPTER 14

GEOTECHNICAL SEISMIC DESIGN

14.1 INTRODUCTION

This Chapter provides guidance for the geotechnical seismic design of bridges, embankments, earth retaining structures (ERS), and miscellaneous structures. Geotechnical seismic design consists of evaluating the Extreme Event I (EEI) limit state of transportation structures owned and maintained by the State of South Carolina for performance under seismic hazards (Chapter 13) and seismic lateral loadings (Chapter 14). Seismic lateral loadings (inertial accelerations) affect soil pressures by increasing static active soil pressures and decreasing static passive soil pressures. Methods for computing seismic active and passive soil pressures are included in this Chapter. The geotechnical seismic design will typically be evaluated using performance based design methodologies to evaluate if the structure's performance meets the geotechnical performance limits presented in this Manual (Chapter 9). Force based design methodologies are included where appropriate for evaluating boundary conditions. If the performance limits are exceeded, seismic hazard mitigation methods will be discussed that can be used to meet the required performance limits.

The procedures for seismic geotechnical design are consistent with those procedures presented for static geotechnical design in this Manual. The seismic geotechnical design guidelines presented in this Manual may not be the only methods available particularly since geotechnical seismic design is constantly evolving and developing. The overall goal of this Chapter is to establish a state-of-practice that can evolve and be enhanced as methodologies improve and regional (CEUS) experience develops. Methods other than those indicated in this Manual may be brought to the attention of the Preconstruction Support Group - Geotechnical Design Section (PCS/GDS) for consideration on a specific project or for consideration in future updates of this Manual.

14.2 GEOTECHNICAL SEISMIC DESIGN APPROACH

Geotechnical seismic design is typically performed using either a Force Based Design or Performance Based Design methodology. SCDOT LRFD geotechnical seismic design of transportation structures typically consists of using Performance Based Design methodologies. The Extreme Event I (EEI) limit state performance limits presented in Chapter 10 should be used as a starting point with significant collaboration between the structural engineer and the geotechnical engineer. The geotechnical seismic design approach will be consistent with design philosophy for structure design. It will be the responsibility of the design team to define the performance of bridges, roadway embankments, and ERSs within the frame work of SCDOT policy of minimizing the susceptibility of a bridge structure to collapse during strong earthquake shaking..

The design approach typically begins by designing the transportation structure for the Strength and Service limit states. The resulting structure is then evaluated for the Extreme Event I (EEI) limit state.

For sites where bridges and bridge foundations are located within soils that are susceptible to soil shear strength loss (SSL) in accordance with Chapter 13 of this Manual due to either cyclic liquefaction of Sand-Like soils or cyclic softening of Clay-Like soils, the design methodology shall include the evaluation of the following two conditions:

No Soil Shear Strength Loss (No Soil SSL) Condition: The structure should first be analyzed and designed for the inertial forces induced by EEI without soil SSL condition. The site response developed should not include any effects of the soil SSL.

Soil Shear Strength Loss (Soil SSL) Condition: The structure, as designed for the No Soil SSL condition, is reanalyzed assuming that the Sand-Like and Clay-Like soils have experienced soil SSL. The soils susceptible to SSL are assigned appropriate residual shear strength in accordance with Chapter 13. The design spectrum for the No Soil SSL is used unless the period of the structure is greater than 1.0 second and the site response is susceptible to increase for the soil SSL condition in accordance with Chapter 12. The geotechnical seismic design is then performed using these soil and site response parameters.

If the structure meets the performance objectives (resistance factors and performance limits) for the EEI limit state, the design is complete. Otherwise, measures to mitigate the effects of the seismic hazard are developed. The traditionally accepted practice to reduce the horizontal acceleration by 50 percent to allow for displacements will not be an acceptable design methodology for SCDOT seismic design practice. Therefore, the full horizontal acceleration ($k_h = k_{avg}$) shall be used in design. Displacements shall be computed in accordance with Chapter 13 when instability is determined. Mitigation can be accomplished by redesign of the structure to resist the seismic hazards (structural mitigation), reducing the effects of the seismic hazard by performing geotechnical mitigation measures, or by developing a mitigation approach that consists of both structural and geotechnical mitigation procedures.

The evaluation of the EEI limit state typically requires a geotechnical evaluation of the Geology and Seismicity (Chapter 11), and the Site Response (Chapter 12) and the evaluation of the effects of the Seismic Hazards (Chapter 13) on the transportation structures being designed. For transportation structures (bridges, retaining walls, etc.) that require structural design of concrete or steel components, the geotechnical engineer typically provides the structural engineer with site response analyses, soil-structure interaction modeling of foundations, seismic loadings (active and passive), and the effects of seismic hazards. When mitigation of seismic hazards is required, the geotechnical engineer provides geotechnical mitigation options and assists in the evaluation of structural mitigation options. When evaluating structural mitigation options, geotechnical engineers may require structural engineering input such as in the design of piles/shafts for providing slope stability of a roadway embankment or river/channel bank at a bridge crossing.

It should be noted that the procedures outlined in this Chapter are to be used with new construction. The seismic design or retrofit of existing structures is currently performed on a case-by-case basis. At the discretion of the Regional Production Engineer (RPE), the Regional Program Manager (PM), or the Regional Design Manager (DM), existing structures may be required to have seismic retrofit design performed.

14.3 SEISMIC LATERAL LOADINGS

Seismic lateral loadings are those seismic hazards that are induced by the acceleration of a soil mass or structure during earthquake shaking. The average seismic horizontal acceleration (k_{avg}) that is generated by the earthquake is computed by using seismic acceleration coefficients as indicated in Chapter 13. The seismic lateral loadings are exhibited as either seismic active soil pressures or seismic passive soil pressures. Seismic active soil pressures are generated when earthquake accelerations mobilize the active soil driving wedge behind an ERS. Seismic passive soil pressures are generated when an earthquake load is applied from a structure to the soil. When passive soil pressures are generated as a result of seismic loadings, the earthquake's inertial acceleration forces also affect the passive soil wedge. The mechanism of this hazard is dependent on the type of structure being analyzed. Because a performance based design is used in the design of transportation structures, the effects of seismic lateral loadings must take into account the added force on the structure and any deformations caused by shearing of the soils. Examples of how this hazard can affect typical transportation structures are provided below:

- ERSs must not only resist static active soil pressure but also seismic active pressures as a result of the average earthquake accelerations acting on the active soil wedge. Additional lateral loads are generated as a result of the acceleration acting on the mass of the retaining structure and any soil that is contained in the structure.
- While a bridge abutment on one end of a bridge may experience the same seismic lateral loadings as ERSs, the abutment at the other end of the bridge may experience seismic induced lateral loads from the bridge and mass of the abutment that places a lateral seismic load on the soils behind the abutment, resulting in passive pressure resistance. Soils retained by bridge abutments will cycle between active and passive soil pressures throughout the earthquake shaking.
- Bridge approach and roadway embankments not only must resist static driving forces, but also seismic driving forces. Seismic driving forces result from peak ground accelerations acting on the soil mass contained within the critical slope failure surface.

14.4 SEISMIC ACTIVE SOIL PRESSURES

Earthquake-induced lateral loadings addressed in this Section are limited to those loadings (seismic active earth pressures) that are the result of soil-structure interaction between soils and ERSs. Seismic active soil pressures are generally analyzed using pseudo-static methods. The pseudo-static method is a force-equilibrium method that is used to analyze external forces and the effects (i.e. sliding, overturning, bearing capacity, etc.) on the structure being designed. The pseudo-static method used to analyze seismic active soil pressures uses the average horizontal acceleration coefficients that has been adjusted for wave scattering, ($k_h = k_{avg}$), multiplied by the weight of the structural wedge (weight of the structure and any soil above the structure) and the weight of the active driving wedge, W_{DW}. The seismic active earth loadings in the pseudo-static method are illustrated in Figure 14-1.

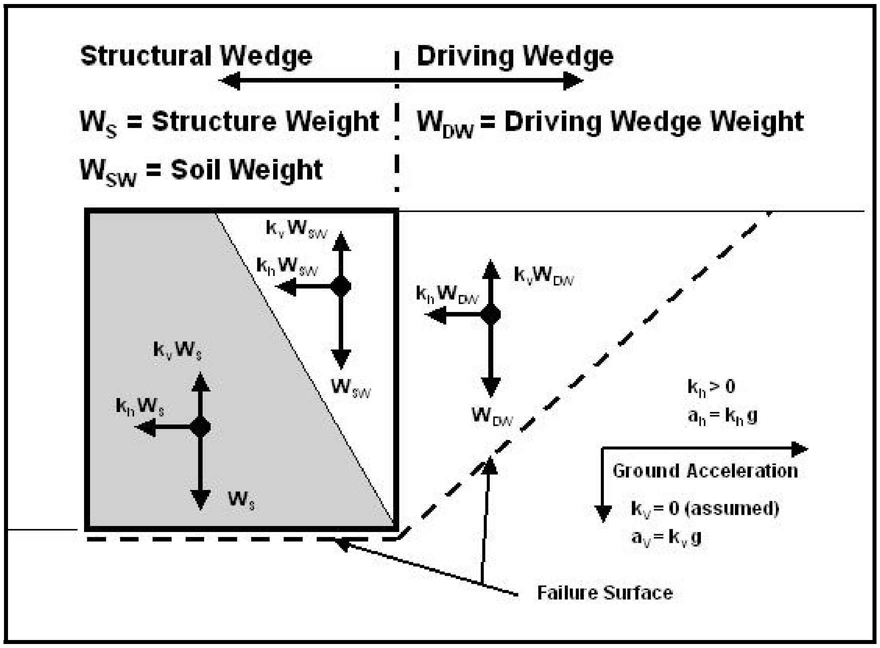


Figure 14-1, Pseudo-Static Method – Inertial Forces and Seismic Loadings (modified Ebeling, et al., 2007)

The limit-equilibrium method is based on the following assumptions:

- The retaining wall yields sufficiently to produce active soil pressures during an earthquake.
- The backfill is a dry cohesionless soil (Mononobe-Okabe method).
- Active failure wedge behaves as a rigid body so that the accelerations are uniform throughout the soil mass.
- The soils behind the wall are not saturated and liquefaction does not occur.

The following methods can be used to evaluate the seismic active pressures:

1. Mononobe-Okabe (MO) Method

2. Trial Wedge Method

3. Seismic Active Pressure Coefficient Charts- NCHRP 611

4. Generalized Limit Equilibrium (GLE) Method

14.4.1 Mononobe-Okabe Method

One of the most frequent methods used to evaluate seismic active loadings is the Mononobe-Okabe (MO) method shown in Figure 14-2. The total dynamic active earth thrust, P_{ae}, is determined by the following equation:

$$P_{ae} = \frac{1}{2}\gamma H^2 (1 - k_v) K_{ae} \qquad \text{Equation 14-1}$$

Where the seismic active earth pressure coefficient, K_{ae}, is determined as follows:

$$K_{ae} = \frac{\cos^2(\phi - \Psi - \theta)}{\cos(\Psi)\cos^2(\theta)\cos(\delta + \theta + \Psi)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \Psi - \beta)}{\cos(\delta + \theta + \Psi)\cos(\beta - \theta)}}\right]^2} \quad \text{Equation 14-2}$$

Where,

γ	=	unit weight of soil
H	=	height of wall or effective height of wall (h_{eff})
ϕ	=	angle of internal friction of soil
Ψ	=	$\tan^{-1}[k_h/(1-k_v)]$
δ	=	angle of friction between soil and wall
k_h	=	horizontal acceleration coefficient
k_v	=	vertical acceleration coefficient, typically set to zero.
β	=	backfill slope angle
θ	=	angle of backface of the wall with the vertical

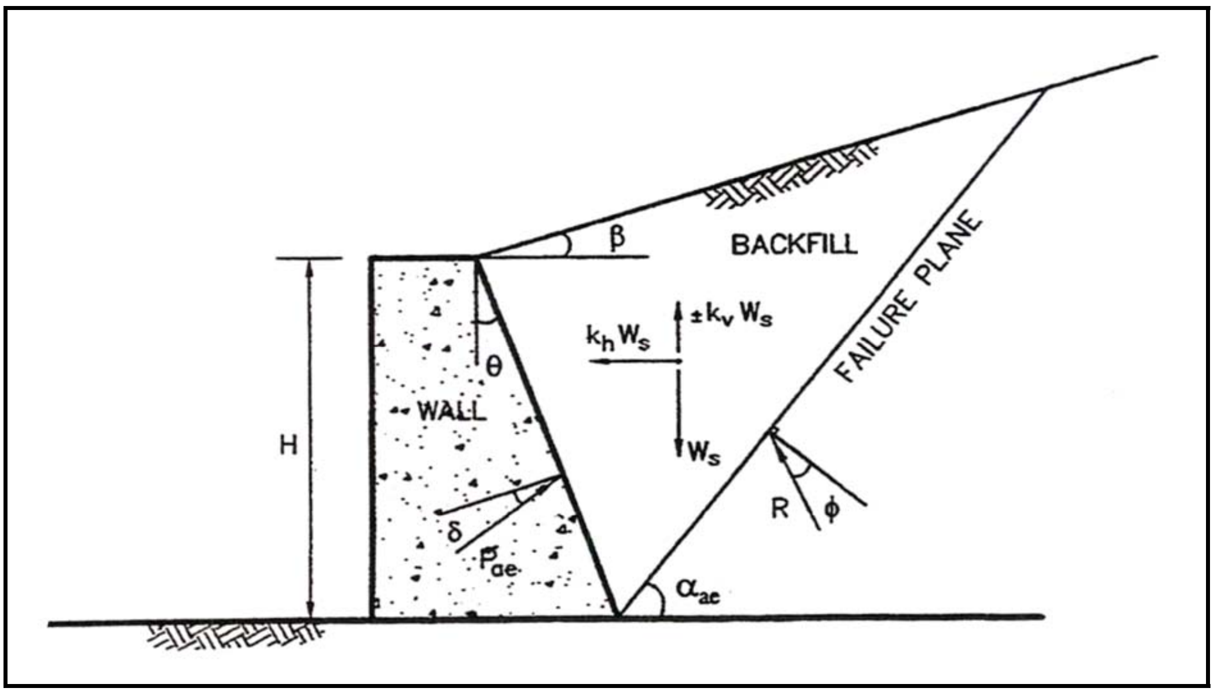


Figure 14-2, Mononobe-Okabe Method
(Munfakh, et. al., 1998)

Although the MO method is often used to compute seismic active soil pressures, this method has been found to produce very high pressures that tend to approach infinity when high accelerations and/or steep backslopes are analyzed. This situation occurs when either of the following limiting conditions are met:

$$\phi \geq \beta + \Psi = \beta + \tan^{-1}\left(\frac{k_h}{1 - k_v}\right) \quad \text{Equation 14-3}$$

$$k_h \leq (1 - k_v)\tan(\phi - \beta) \qquad \text{Equation 14-4}$$

Because the MO equation is limited to backfills consisting of dry cohesionless soils that may not typically be found at very large distances behind the wall, the MO method may not be the most appropriate analytical method.

Because of the various limitations associated with the MO equation, the use of this equation should be limited to the following criteria provided that the limiting conditions of Equations 14-3 and 14-4 are met:

- Backfill slopes, $\beta \leq 18.4$ degrees (3H:1V or flatter)
- Limited to $K_{ae} \leq 0.60$
- Free draining backfill materials (cohesionless soils) behind the wall should extend throughout the seismic active wedge.

14.4.2 Trial Wedge Method

The Trial Wedge method can be used to determine critical earthquake-induced active forces when the MO method is not the appropriate method. The trial wedge method is more adaptable and can accommodate various types of soil behind the wall and relatively complex surface geometries. The Trial Wedge method is illustrated in Figure 14-3.

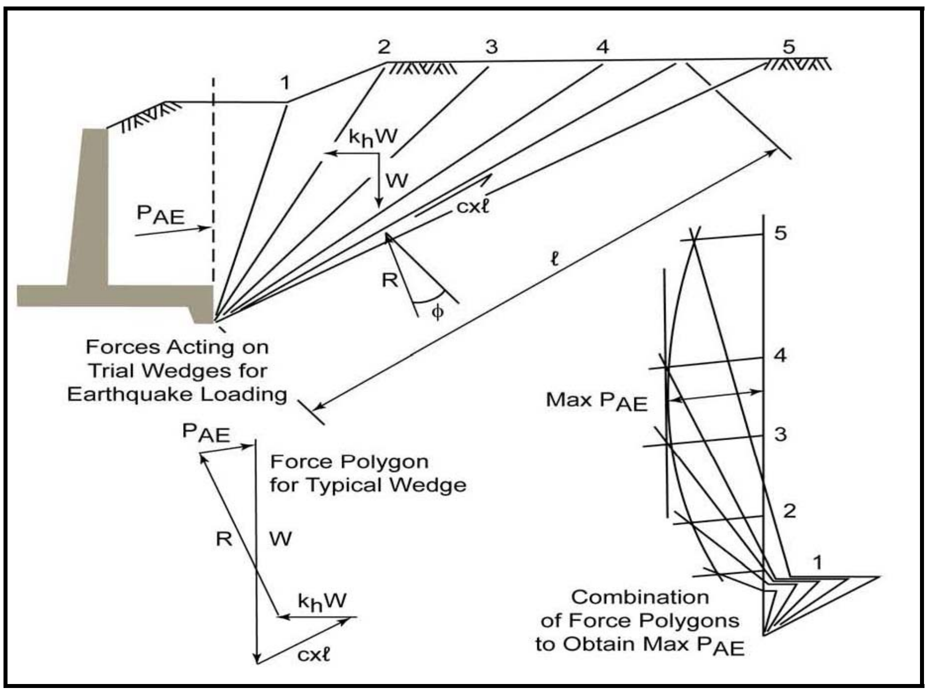


Figure 14-3, Trial Wedge Method (Anderson, et. al., 2008)

Details on conducting the trial wedge method of analysis can be found in Ebeling et al (2007) and Bowles (1982). It should be noted that the earthquake-induced inertial forces resulting from the structural wedge (retaining structure or soil mass contained within the structural wedge), as indicated in Figure 14-1, are not included in the MO method or the Trial Wedge method and that the structural wedge must also be included in the analysis.

When the horizontal acceleration, k_h, is equal to the peak horizontal ground acceleration (PGA), the seismic active loadings can become very large resulting in the design of the retaining structure becoming increasingly large and uneconomical. Designing for k_h = PGA will limit the deformations to zero. If deformations can be tolerated within the performance limits of the structure, then the horizontal acceleration, k_h, can be reduced. The method of reducing the horizontal acceleration, k_h, consists of allowing displacements to occur as provided in the Chapter 13.

14.4.3 Seismic Active Pressure Coefficient Charts

NCHRP 611 (Anderson et al., 2008) has developed the following Coulomb-type wedge analysis that is based on the trial wedge method as shown in Figure 14-4. The following equation allows the input of cohesion into developing the seismic active earth pressure (P_{ae}):

Equation 14-5

$$\boldsymbol{P_{ae}} = \frac{\boldsymbol{W}[(\mathbf{1}-\boldsymbol{k_v})\mathbf{tan}(\alpha-\phi)+\boldsymbol{k_h}]-\boldsymbol{cL}[\mathbf{sin}(\alpha)\mathbf{tan}(\alpha-\phi)+\mathbf{cos}(\alpha)]-\boldsymbol{c_a H}[\mathbf{tan}(\alpha-\phi)\mathbf{cos}(\omega)+\mathbf{sin}(\omega)]}{[\mathbf{1}+\mathbf{tan}(\delta+\omega)\mathbf{tan}(\alpha-\phi)]^{\mathbf{cos}(\delta+\omega)}}$$

Where,

α	=	failure plane angle (Variable)
ϕ	=	angle of internal friction of soil
k_h	=	average horizontal acceleration coefficient adjusted for wave scattering
k_v	=	vertical acceleration coefficient, typically set to zero.
c	=	soil cohesion
c_a	=	soil wall adhesion
δ	=	angle of friction between soil and wall (δ = 0.67φ)
ω	=	wall angle
H	=	height of wall
L	=	L_n = Length of failure surface AH located along failure plan angle (α)
W_1	=	weight of wedge ABCDEF + q_1 + f
W_n	=	weight of wedge ABCDEGH + q_1 + q_2 + f
W_{n+1}	=	weight of wedge ABCDEGI + q_1 + q_2 + f
q_1	=	uniform strip surcharge located between DE
q_2	=	uniform strip surcharge located between GI
f	=	line load located between BC

The design parameters should be selected based on site conditions. The only parameter that must be determined on a trial basis is the failure plane angle (α_n). The failure plane angle (α_n) is determined by varying the failure plane angle (α_n) until the maximum P_{an} = P_{ae} is computed.

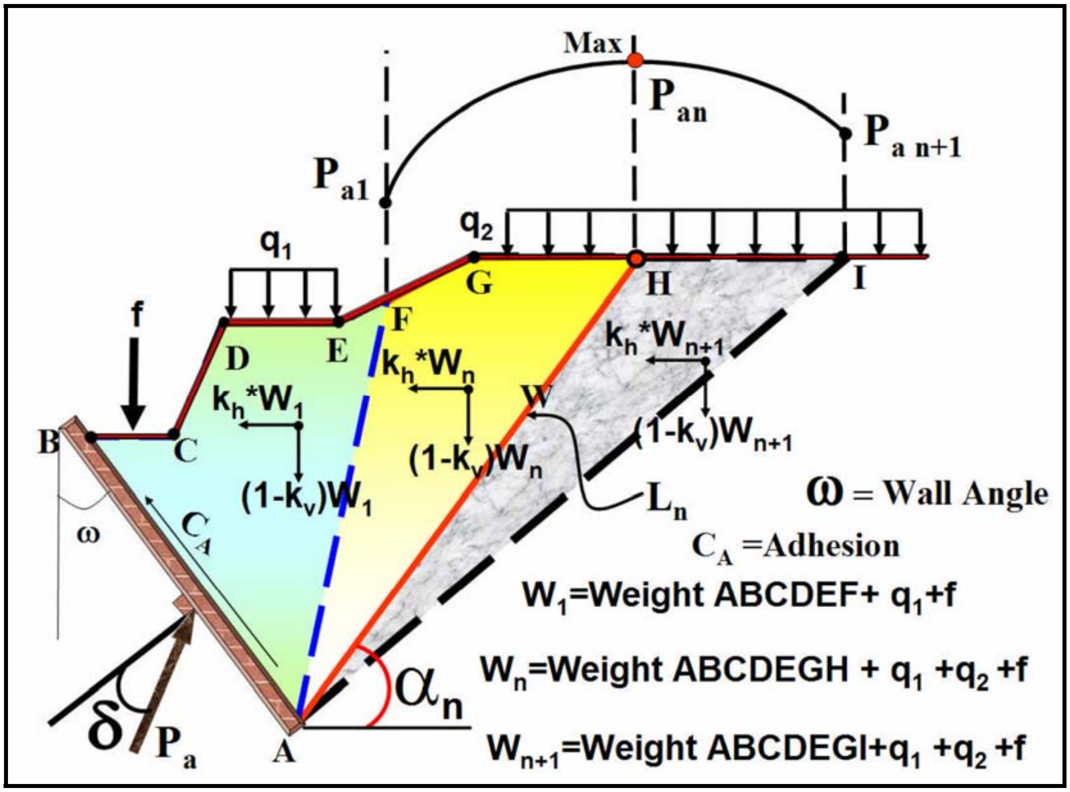


Figure 14-4, Active Seismic Wedge
(Anderson, et. al., 2008)

Design charts shown in Figures 14-5, 14-6, and 14-7 have been developed based on:

1. Level ground behind the wall
2. Wall friction angle, $\delta = 0.67\phi$
3. Angle of internal friction, $\phi = 30°$, $35°$ and $40°$
4. Shear strength friction ratios of $C/\gamma H$ = 0.0, 0.05, 0.10, 0.15, 0.20, 0.25, and 0.30 where:

   C = cohesion

   γ = unit weight of soil

   H = wall height

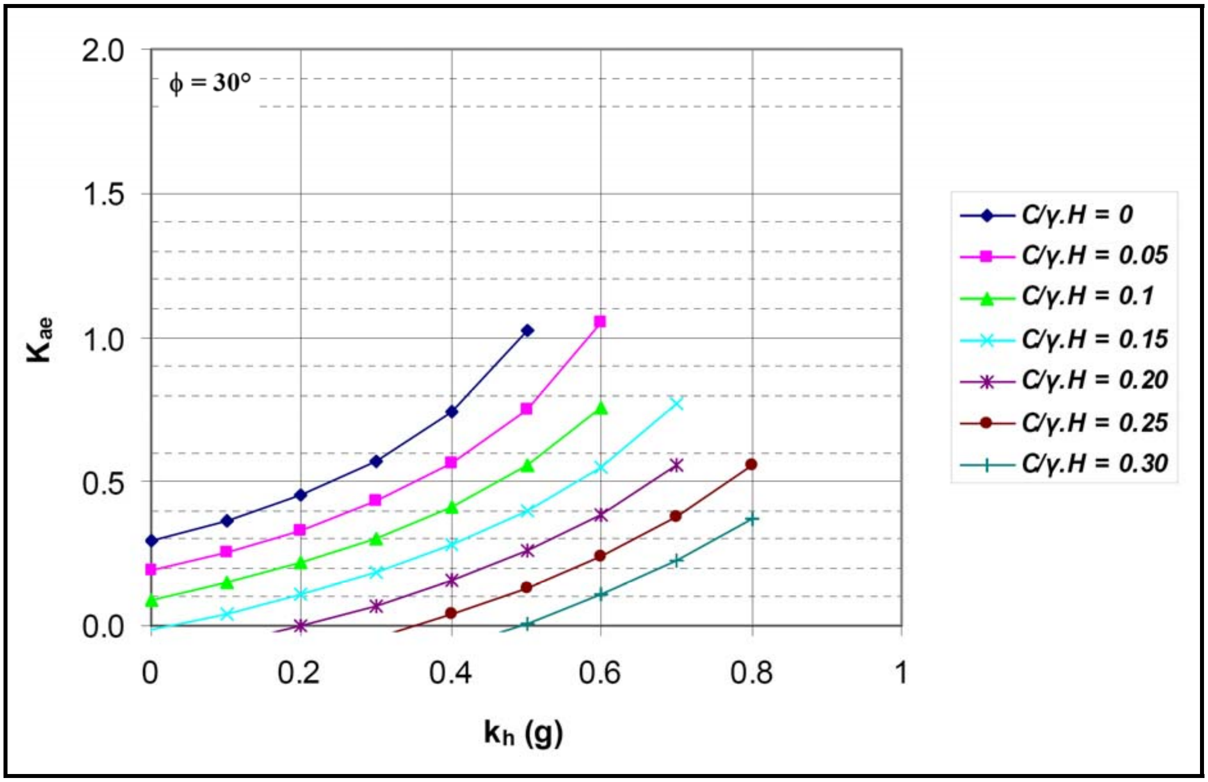


Figure 14-5, Seismic Active Earth Pressure Coefficient (φ = 30°)
(Anderson, et. al., 2008)

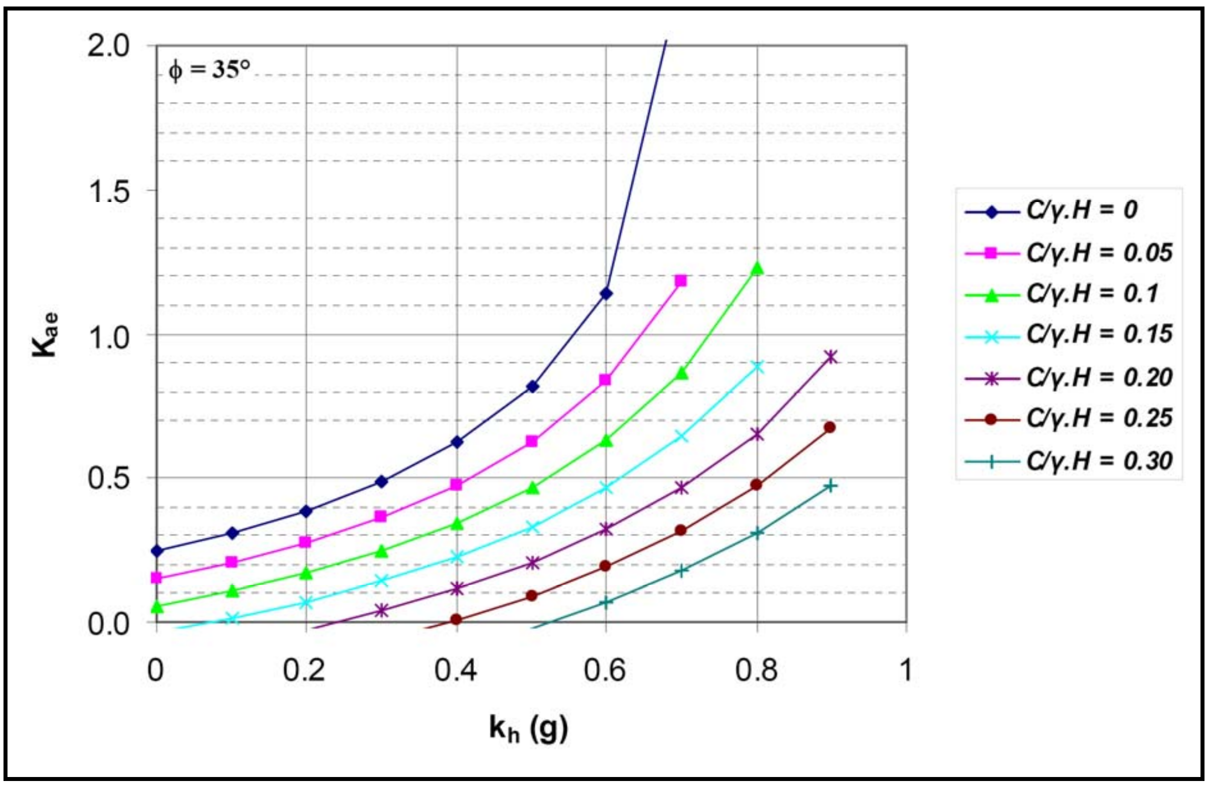


Figure 14-6, Seismic Active Earth Pressure Coefficient (φ = 35°)
(Anderson, et. al., 2008)

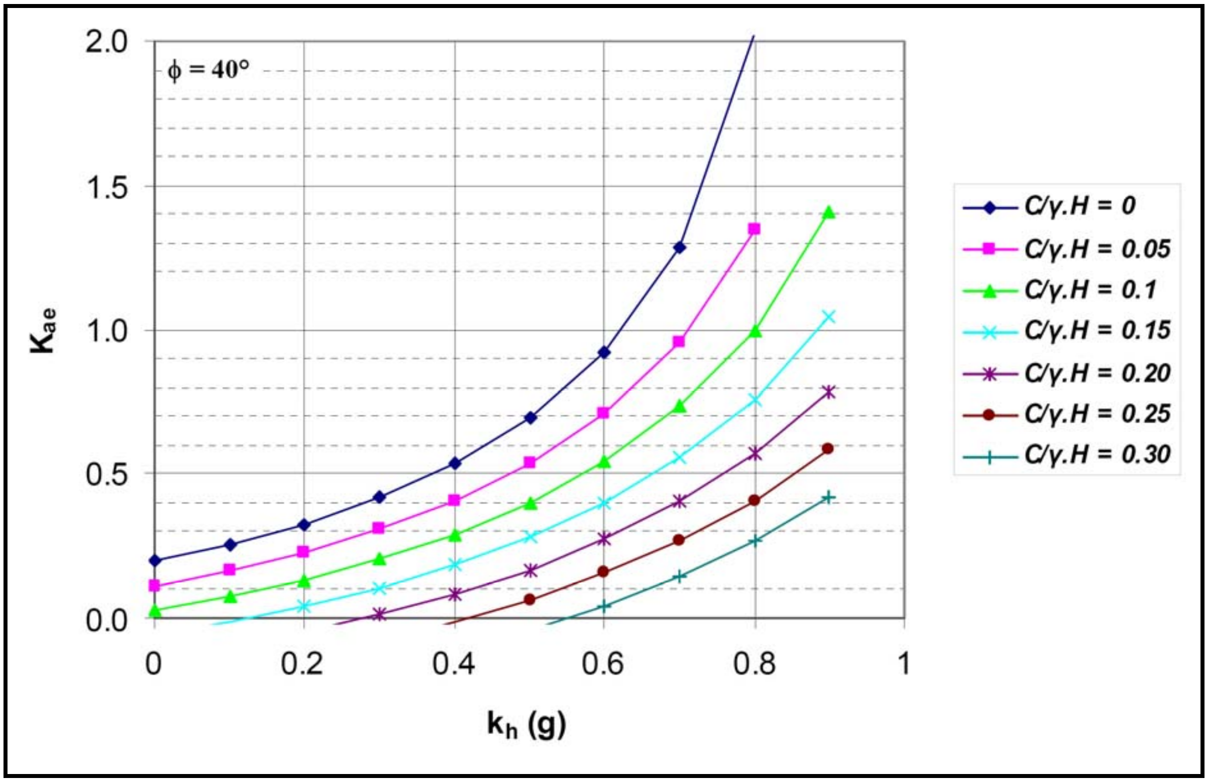


Figure 14-7, Seismic Active Earth Pressure Coefficient (φ = 40°)
(Anderson, et. al., 2008)

14.4.4 Generalized Limit Equilibrium (GLE) Method

The generalized limit equilibrium (GLE) method can also be used to evaluate critical earthquake-induced active forces when the MO method is not the appropriate method. This method has been included in several conventional limit-equilibrium slope stability computer programs. This method is the most robust of the limit equilibrium methods because it can handle complex geometries, it incorporates various soil layers, and it allows the user to explore unlimited failure surfaces and soil combinations without sacrificing time or accuracy.

The slope stability method that should be used in this analysis is Spencer's method because it satisfies the equilibrium of forces and moments. Circular, linear, multi-linear, or random failure surfaces should be investigated. Additional guidance on using this method can be obtained from NCHRP Report 611 by Anderson, et. al. (2008) and Chugh (1995).

14.4.5 Unyielding Structures

Seismic active soil pressures require that the wall yield sufficiently to mobilize minimum active soil pressures. Approximate values of relative movement required to reach active earth pressure conditions are indicated in Table 14-1 and graphically displayed for granular soils in Figure 14-8.

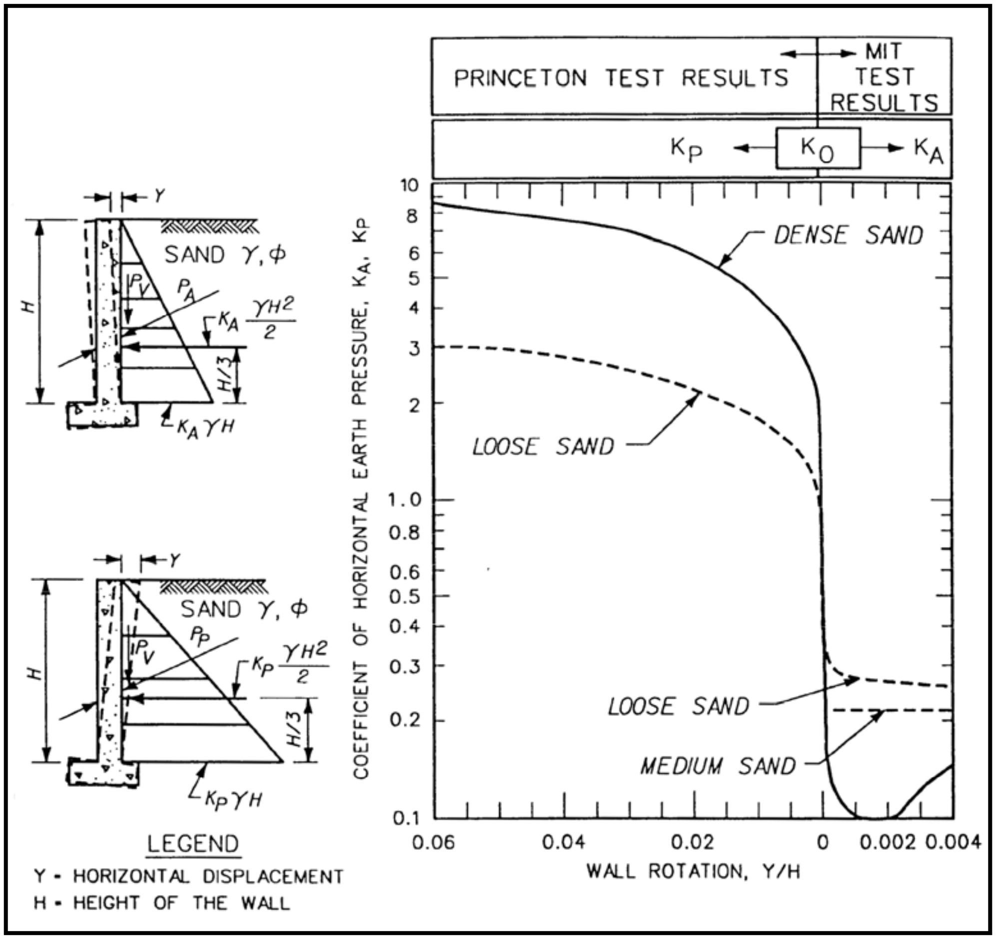


Figure 14-8, Effects of Wall Movement on Static Horizontal Earth Pressures (NAVFAC DM-7.2, 1986)

Table 14-1, Relative Movements Required To Reach Active Earth Pressures (Clough and Duncan, 1991)

Type of Backfill	Δ / H
Dense Sand	0.001
Medium Dense Sand	0.002
Loose Sand	0.004
Compacted Silt	0.002
Compacted Lean Clay	0.010
Note: Δ = movement of top of wall (feet); *H* = height of wall (feet)	

If the retaining structure is rigidly fixed or restrained from movement, the earthquake-induced forces will be much higher than those predicted by the seismic active pressures. Analytical methods to evaluate these types of loads on unyielding structures are not within the state of practice for this type of structure. AASHTO requires that walls and abutments that are rigidly fixed or restrained from movements be designed using horizontal acceleration coefficients that are 1.5 times the horizontal acceleration coefficient (k_h). The average horizontal acceleration coefficient (k_h) that has been adjusted for wave scattering should be used when computing seismic lateral loadings on unyielding structures. The vertical accelerations shall be set to zero.

14.5 SEISMIC PASSIVE SOIL PRESSURES

Earthquake-induced lateral loadings can mobilize passive soil pressure resistance such as those that occur when an ERS resists sliding by either shear keys or when an abutment backwall is displaced into the backfill. The passive resistance versus displacement behind bridge abutment wall is provided in Section 14.9.

For retaining wall components subject to passive resistance that are less than 5 feet in height, the passive resisting forces shall be computed using static passive forces. Static passive forces for wall heights or foundation thicknesses less than 5 feet shall be used because it is anticipated that the inertial effects from earthquake loadings will be small (see Chapter 18).

For retaining wall components subject to passive resistance that are 5 feet or greater in height, the passive resisting forces must take into account the inertial effects from the earthquake. The MO method of determining passive pressure coefficients shall not be used due to the various limitations of the method. The seismic passive earth pressure coefficient (K_{pe}) shall be obtained using the Shamsabadi (2006) and Shamsabadi et al., (2007) method presented in NCHRP 611 by Anderson et al. (2008), which accounts for wall friction and nonlinear failure surface within the soil. The seismic passive earth pressure coefficient (K_{pe}) for a soil with cohesion and internal friction using a log spiral procedure is computed using the following equations:

$$dE_i = \frac{W_i(1-k_v)[\tan(\alpha_i+\phi)-k_h]+cL_i[\sin(\alpha_i)\tan(\alpha_i+\phi)+\cos(\alpha_i)]}{[1-\tan(\delta_i)\tan(\alpha_i-\phi)]^{\cos(\delta)}} \quad \text{Equation 14-6}$$

$$P_{PE} = \frac{\sum_1^i dE_i}{[1-\tan(\delta_w)\tan(\alpha_i-\phi)]^{\cos(\delta_w)}} \quad \text{Equation 14-7}$$

$$K_{PE} = \frac{2P_{PE}}{\gamma h^2} \quad \text{Equation 14-8}$$

Where the following parameters are defined in Figure 14-9,

ϕ	=	angle of internal friction of soil
k_h	=	average horizontal acceleration coefficient adjusted for wave scattering (K_h in Figure 14-9)
k_v	=	vertical acceleration coefficient, typically set to zero (K_v in Figure 14-9)
c	=	soil cohesion
δ_w	=	angle of friction between soil and wall (δ = 0.67 φ)
δ_i	=	angle of friction between interslice
α_i	=	angle at base of slice
E	=	sum of interslice horizontal force (passive resistance)
dE_i	=	increase in interslice horizontal resistance force (passive resistance)
T	=	sum of interslice shear force
dT	=	increase in interslice shear force
W_i	=	weight of soil slice
P_{PE}	=	Seismic Passive Earth Pressure

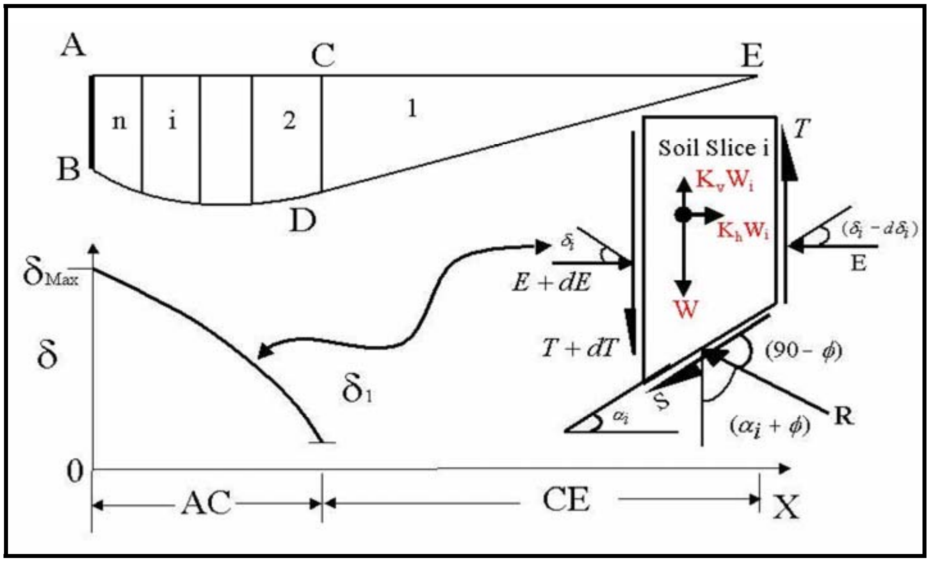


Figure 14-9, Limits and Shape of Seismic Interslice Force Function (Shamsabadi, 2006)

This method of analysis divides the sliding mass of the backfill into many slices. It is assumed that the shear forces dissipate from a maximum (δ_{max}) at the wall face (AB) to the induced seismic shear forces at the face (CD) of the first slice as shown in Figure 14-9.

Shamsabadi (2006) and Shamsabadi et al. (2007) have developed charts based on a log spiral procedure to determine the seismic passive pressure coefficient (K_{pe}) based on a wall friction angle (δ) of 0.67 times the soil friction angle (ϕ), a range of average seismic horizontal coefficients (k_h), effective soil friction angles (ϕ'), and effective soil cohesion shear strength ratios ($c'/\gamma H = c/\gamma H$). These charts are presented in Figures 14-10, 14-11, and 14-12.

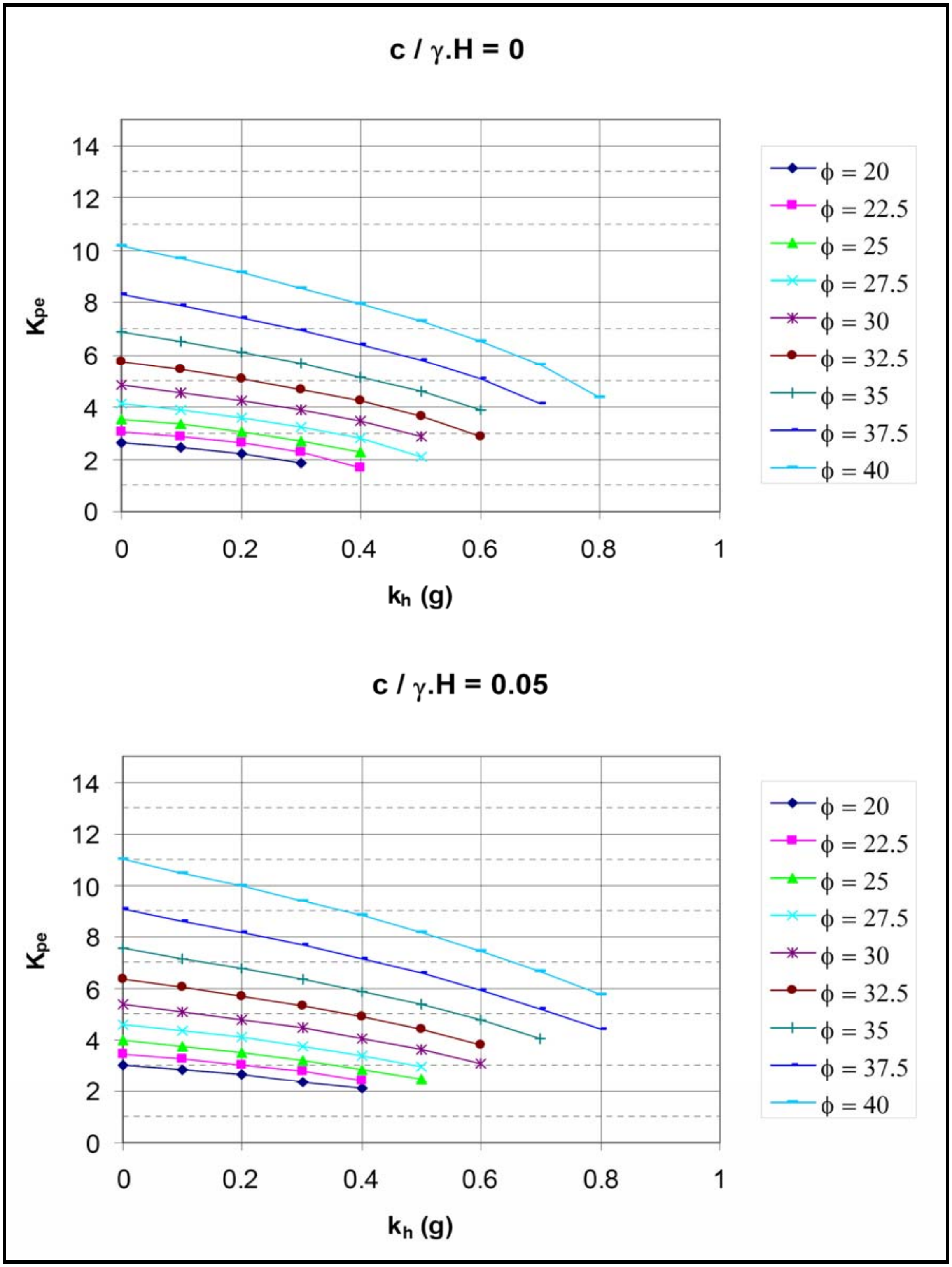


Figure 14-10, Seismic Passive Earth Pressure Coefficient (c'/γH = 0 & 0.05) (Shamsabadi, 2006)

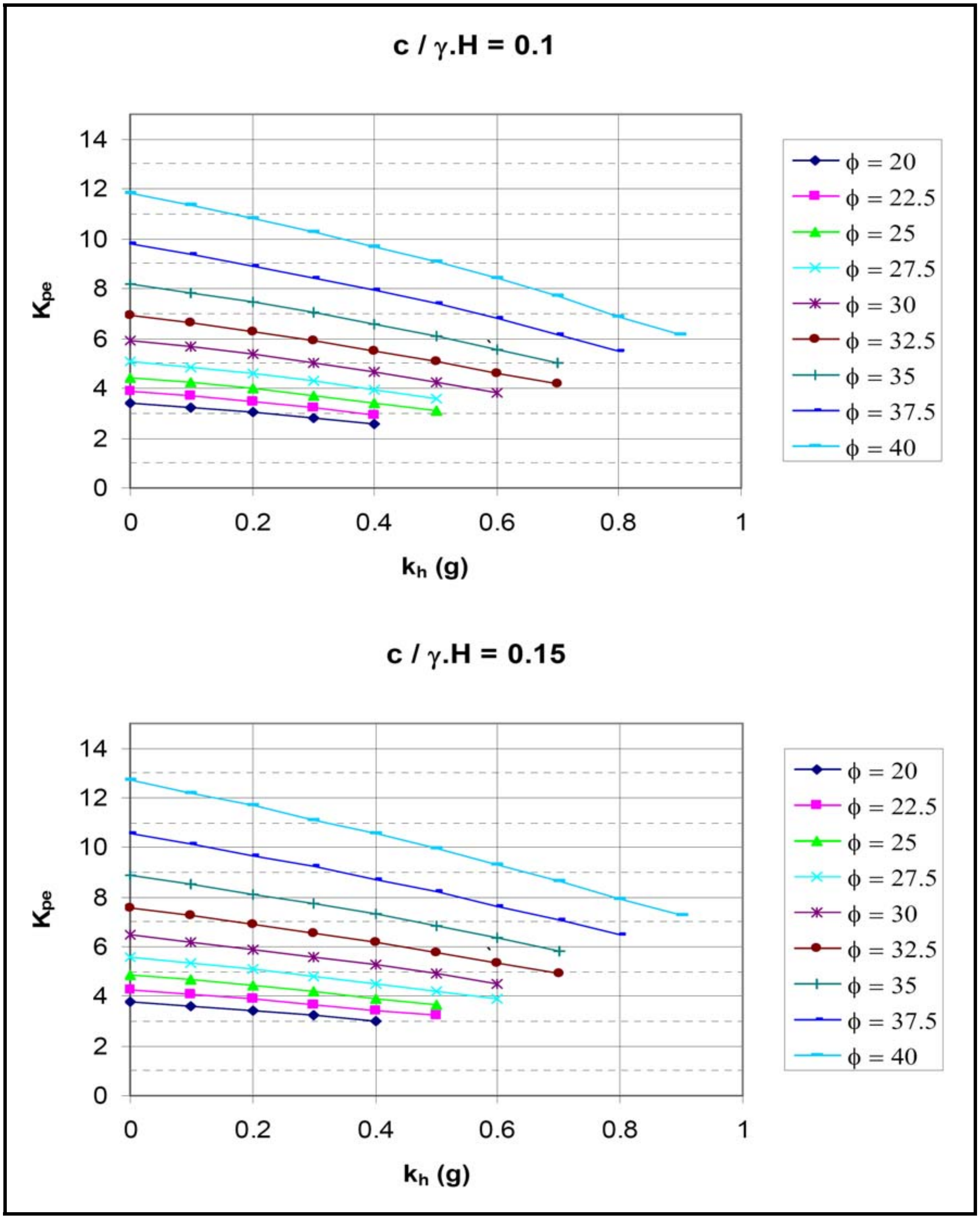


Figure 14-11, Seismic Passive Earth Pressure Coefficient (c'/γH = 0.10 & 0.15) (Shamsabadi, 2006)

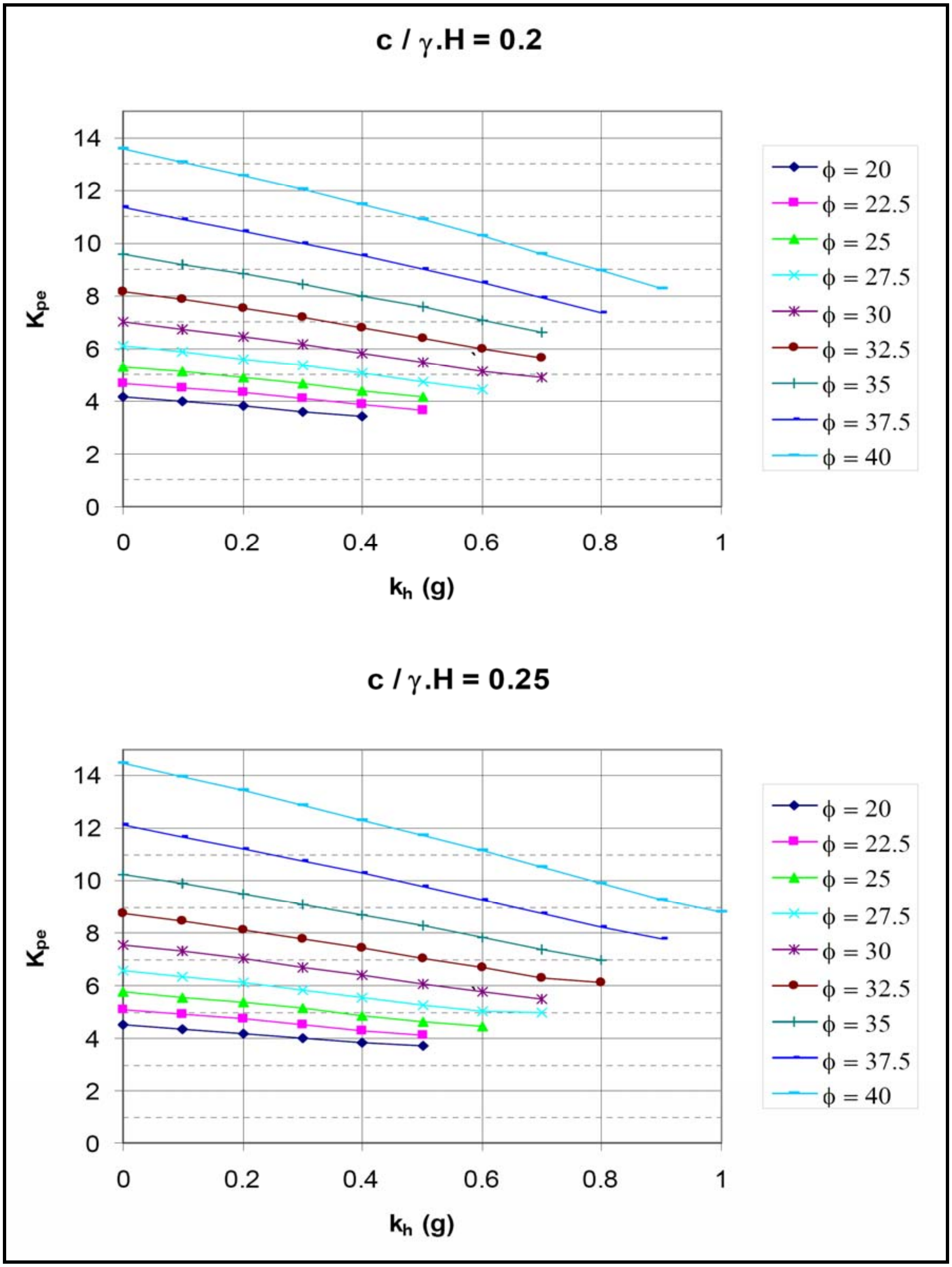


Figure 14-12, Seismic Passive Earth Pressure Coefficient (c'/γH = 0.20 & 0.25) (Shamsabadi, 2006)

14.6 GEOTECHNICAL SEISMIC DESIGN OF BRIDGES

The geotechnical seismic design of bridges is a collaborative effort between the geotechnical engineer and the structural engineer. In order to provide the appropriate geotechnical seismic design information, the geotechnical engineer will need to develop an understanding of the bridge design and behavior under seismic loading. The geotechnical engineer will need to become familiar with:

- Bridge Characteristics: structural fundamental period (T_F), structure type, bridge damping (i.e. 5%), and bridge plans.
- Structural analysis method to be used by the structural engineer to model the bridge foundations and abutments.
- Hydraulic Criteria: For the FEE design event, use the full scour from the design flood (100-year event). For the SEE design event, use half of the the scour obtained from the design flood (100-year event) unless otherwise specified by SCDOT.
- Performance Criteria: Geotechnical seismic design for the Extreme Event I (EEI) design uses a Performance Based Design methodology. It is, therefore, necessary to establish performance criteria that are specific to the bridge being designed. The performance limits provided in Chapter 10 should be used as a guide. Performance limits may need to be revised as the bridge design is modified to accommodate bridge movements.

The geotechnical engineer typically provides the structural engineer the following:

- Acceleration Design Response Spectrum (ADRS) Curves
- Bridge Abutment Soil-Structure Interaction Boundaries
- Foundation Soil-Structure Interaction Boundaries
- Effects of Seismic Hazards on the Bridge
- Geotechnical mitigation options to eliminate or reduce the effects of seismic hazards

14.6.1 Acceleration Design Response Spectrum (ADRS) Curves

The site response for the design earthquake (FEE or SEE) is represented by a horizontal Acceleration Design Response Spectrum (ADRS) curve that represents the pseudo-spectral accelerations for the uniform hazard at different frequencies or periods. The geotechnical engineer develops ADRS curves at either the ground surface or the depth-to-motion location of the bridge element being evaluated as presented in Chapter 12 - Earthquake Engineering. The site response is evaluated by either using the Three-Point method or performing a Site-Specific Seismic Response Analysis.

The Site-Specific Response Analysis is typically performed using one-dimensional equivalent linear site response software (i.e. SHAKE91). When the subsurface site conditions and earthquake motion input exceed the limitations of the one-dimensional equivalent linear site response methodology as indicated in Chapter 12, a non-linear site response analysis using appropriate non-linear site response software (i.e. DMOD) must be used to develop the ADRS.

A minimum of seven earthquake motions (time histories) may be used in order to capture the site response when performing a Site-Specific Seismic Response Analysis. When seven earthquakes are used, the site-response spectral accelerations shall be averaged. However, if the procedures outlined in Chapter 12 are used (i.e. three earthquake motions), then an envelope of all three site-response spectral accelerations shall be required. All earthquake input motions must be scaled to match the uniform hazard spectral accelerations. SCDOT typically provides the earthquake input motion by developing synthetic earthquake time histories. The software used to develop the synthetic earthquake input motions can vary the frequency content by using different seeds. The use of real strong motion earthquakes will require peer review and approval by the PCS/GDS. If less than seven earthquakes are used, the site-specific response spectral accelerations curve shall envelope the site response curves for all earthquakes evaluated.

A bridge site can have multiple site response curves depending on the subsurface soils, depth-to-motion of the foundations (i.e. interior bents vs. bridge abutments), fundamental period of the bridge, and the number and locations of joints on the bridge (i.e. is the bridge jointless, regardless of length or does the bridge have a number of joints that have ability to absorb deflections). The development of a single ADRS curve for use in bridge seismic analyses will require input from the structural engineer. The structural engineer will provide input as to the site response curve that will have the largest effect on the behavior of the bridge during a seismic event. For jointless bridges and those bridges without sufficient ability to absorb deflections, the site response curve generated at the bridge abutment typically has the largest impact on the seismic design of a bridge and may be used as the ADRS curve with concurrence from the structural engineer. For bridges with sufficient ability to absorb deflections, it may be necessary to develop an ADRS curve that envelopes all of the site-response curves for the bridge site. This necessitates that the geotechnical and structural engineers work together to evaluate the anticipated structure performance.

14.6.2 Bridge Abutments

The geotechnical engineer needs to be familiar with the different types of bridge abutments that are currently being used by SCDOT and the effect of the seismic demand on the performance of the bridge abutment. The BDM identifies the following three types of bridge abutments:

- **Free-Standing End Bent (See Figure 14-13):** The free-standing end bent has the bridge girder supported on expansion bearings and is not in contact with the backwall. The movements of the bridge during a seismic event can be accommodated by either providing a wider expansion joint and a wider seat supporting the bridge girders or by allowing the backwall to break at the interface with the bent cap.
- **Semi-Integral End Bent (See Figure 14-14):** The semi-integral end bent has the bridge girder supported on expansion bearings and the girder is embedded into the backwall. The backwall is separated from the bent cap by use of a bond breaker between the two structural elements. This allows the structural engineer to use passive pressure of the soils behind the backwall only to resist seismic longitudinal forces.
- **Integral End Bent (See Figure 14-15):** The integral end bent has the bridge girder supported on a bearing plate/neopreme pad and the girder is embedded into the backwall. The backwall is designed to be integral with the bent cap. This allows the structural engineer to use passive pressure of the soils behind the backwall and bent cap to resist seismic longitudinal forces. Additional resistance to seismic longitudinal forces is obtained from the lateral resistance to displacement of the end bent abutment deep foundations (i.e. piles or drilled shafts). The use of passive soil pressure

resistance and the lateral deep foundation resistance to seismic longitudinal forces will require that these resistant forces maintain strain compatibility.

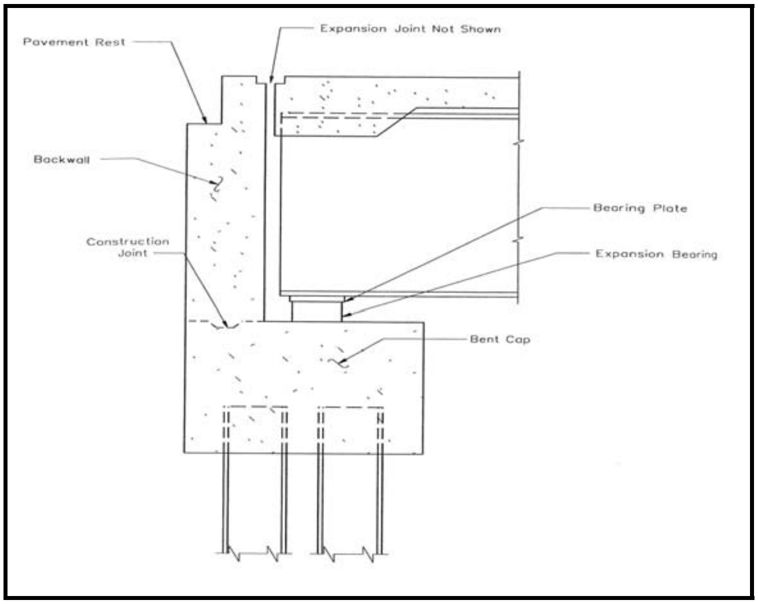


Figure 14-13, Free-Standing End-Bent (SCDOT, 2006)

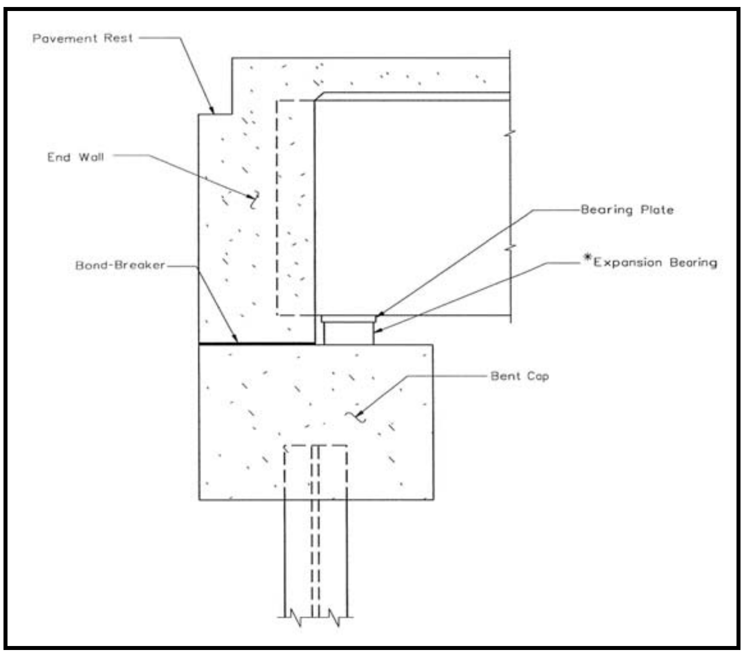


Figure 14-14, Semi-Integral End-Bent (SCDOT, 2006)

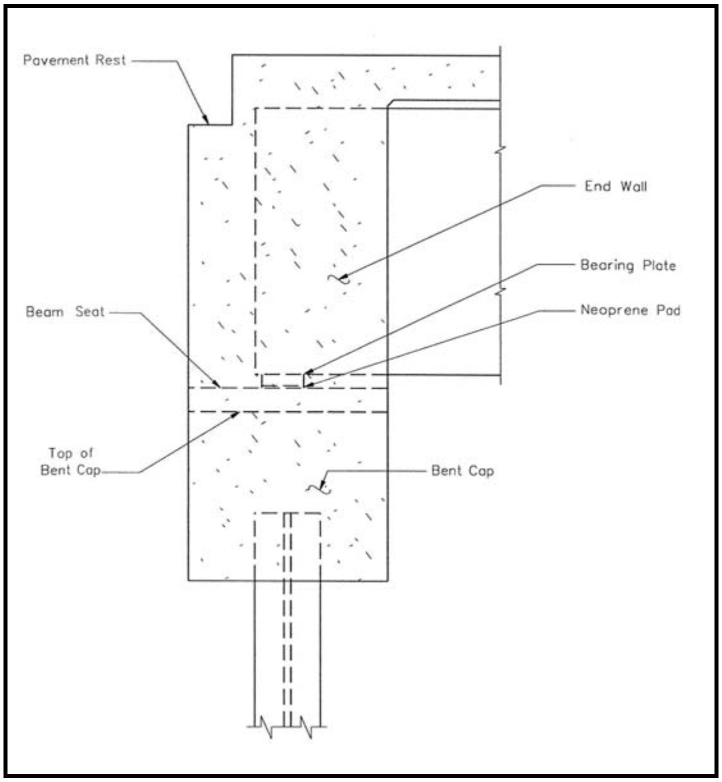


Figure 14-15, Integral End-Bent
(SCDOT, 2006)

The geotechnical engineer will provide soil-structure design parameters of the bridge abutment to the structural engineer. Soil-structure design parameters will be developed based on the structural engineer's modeling requirements and abutment performance. Soil-structure parameters typically require either a single lateral linear spring to be used for the entire bridge abutment or a matrix of linear and rotational springs in all principal directions (i.e. *x*, *y*, and *z*). Because soil-structure interaction is typically non-linear, the secant modulus of linear springs must be provided to be compatible with the displacements. An analysis using the secant modulus typically requires several iterations on spring stiffness and displacement until the two parameters converge.

14.6.3 Bridge Approach Embankment

Bridge approach embankment is typically defined as the roadway embankment located within 150 feet of the bridge abutment. The bridge approach embankment is designed to meet performance objectives of the bridge abutment by using performance limits for the appropriate roadway operational classification (ROC) that is based on the bridge operation classification (OC). The bridge approach embankments are, therefore, designed for more stringent performance limits than are typically used for roadway embankments (ROC=III).

14.6.4

Bridge Foundations

The performance of a bridge structure that is subject to earthquake shaking is dependent on the superstructure and substructure (bridge foundations). Bridge foundations are typically driven piles or drilled shafts.

The geotechnical engineer is responsible for providing the bridge foundation's soil-structure interaction model in order for the structural engineer to be able to evaluate the performance of the bridge due to the seismic demand. Additional seismic design requirements are presented for shallow foundations in Section 14.7 and for deep foundations in Section 14.8.

14.7 SHALLOW FOUNDATION DESIGN

Shallow foundations shall be designed for EEI loads using the procedures outlined in Chapter 15 of this Manual. Shallow foundations are never considered when the subsurface sand-like soils are identified to be susceptible to cyclic liquefaction as defined in Chapter 13. All of the limitations provided in Chapter 15 for shallow foundations shall apply to the use of shallow foundations during EEI. Any settlement induced by the EEI shall be determined in accordance with procedures indicated in Chapter 13 of this Manual.

14.8 DEEP FOUNDATION DESIGN

The geotechnical engineer typically assists the structural engineer in modeling the foundation performance. The performance of deep foundations is typically modeled by evaluating the soil-structure interaction between the deep foundation (i.e. driven piles, drilled shafts, etc.) and the subsurface soils. The soil-structure interaction is dependent on maintaining compatibility between the response of the deep foundation and the response of the soil when evaluating axial and lateral loads.

Geotechnical seismic hazards (Chapter 13) can significantly impact deep foundations and consequently, the performance of the bridge. The failure mechanism of these geotechnical hazards needs to be thoroughly evaluated and understood in order to consider the effects of the geotechnical seismic hazard on deep foundations supporting a bridge. Deep foundation failure mechanisms are presented in Figure 14-16.

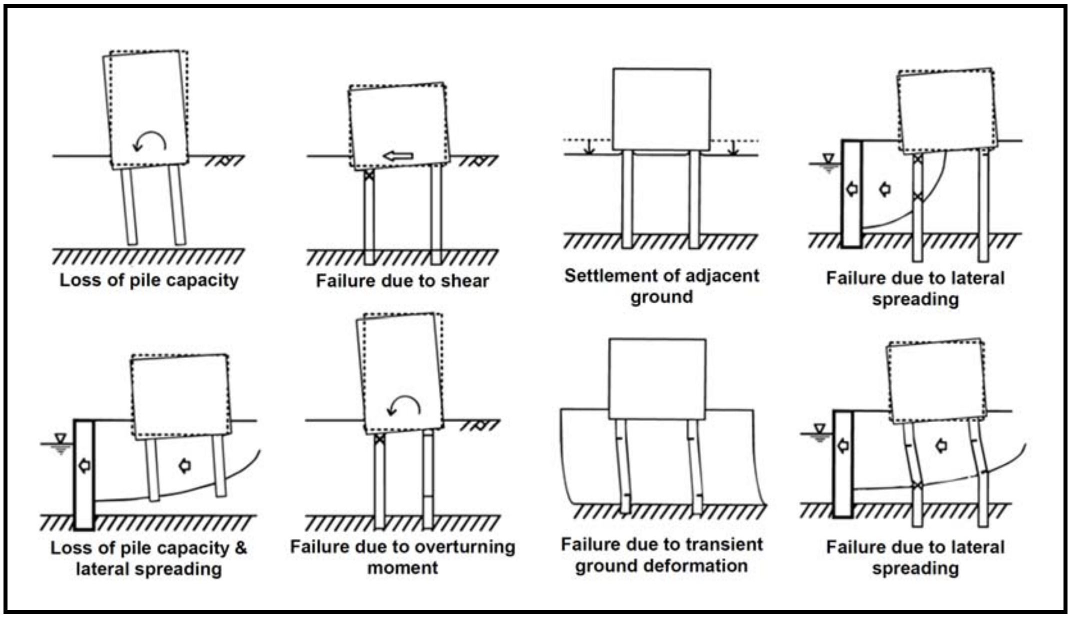


Figure 14-16, Pile Damage Mechanisms in Liquefied Ground (Boulanger et al., 2003)

Geotechnical seismic hazards such as soil shear strength loss (SSL) and seismic settlement can reduce the axial bearing capacity of the deep foundation (Section 14.8.1) and the lateral soil resistance (Section 14.8.3). Deep foundation axial bearing capacity can be further reduced by down drag loads (Section 14.8.2) induced during seismic settlement of the subsurface soils. Seismic hazard displacements due to flow slide failure, lateral spreading, and seismic global instability may impose lateral soil loads on the deep foundations as discussed in Section 14.8.4. The lateral soil loads on the deep foundations will increase the complexity of the soil-structure interaction. The effects of the lateral soil loads on soil-structure interaction between the substructure (i.e. footings, single deep foundations, deep foundation groups, etc.) are discussed in Section 14.8.5. The effects of geotechnical seismic hazards may significantly impact the performance of the bridge and may require mitigation as discussed in Section 14.15.

14.8.1 Axial Loads

When soil SSL is anticipated based on Chapter 13, the axial capacity of deep foundations for the EEI limit state shall be evaluated by adjusting the soil shear strength properties to residual soil shear strength in accordance with Chapter 13 and computing the axial capacity using the methods presented in Chapter 16. If the subsurface soils are susceptible to seismic settlement (Chapter 13), the axial capacity shall be evaluated using downdrag loads as indicated in Section 14.8.2.

14.8.2 Downdrag Loads

Geotechnical seismic hazards such as seismic soil settlement can induce downdrag loads on deep foundations similarly to downdrag loads that result from soil consolidation. Seismic soil settlements in unsaturated soils can occur as a result of densification or seismic compression. Seismic settlement of saturated soils is typically due to densification of Sand-Like soils that are subject to cyclic liquefaction.

Soils experiencing seismic settlement and those soils above the depth of seismic settlement will induce downdrag loads on deep foundations. Analytical methods for evaluating downdrag on deep foundations are provided in Chapter 16. Downdrag loads induced from unsaturated soils and soils not subject to seismic settlement should be based on soil-pile adhesion developed from total peak soil shear strengths. The shear strength of Sand-Like soils during cyclic liquefaction will initially be reduced to liquefied shear strength (τ_{rl}) as the soil reaches full liquefaction (Excess pore pressure, ratio $R_u \approx 1.0$). As the pore pressure dissipates ($R_u < 1.0$), seismic soil settlement occurs and at some point, the soil shear strength will begin to increase and the soils will be in a state of limited liquefaction. The soil-pile adhesion of Sand-Like soils during cyclic liquefaction is, therefore, greater than the liquefied shear strength (τ_{rl}), but considerably less than the undrained peak soil shear strengths (τ_{Peak}). Therefore, the selection of the soil shear strength of Sand-Like soils during cyclic liquefaction should be based on using the soil shear strength occurring during limited liquefaction (i.e. R_u = 0.4) in accordance with Chapter 13.

14.8.3 Lateral Soil Response of Liquefied Soils (P-Y Curves)

Lateral resistance of deep foundations and cantilever retaining wall systems are typically modeled by "non-linear springs" that represent the lateral soil resistance and deflection response (p-y curves). The lateral response of liquefied soils consists of estimating the lateral resistance of the liquefied soils (p_L) and the corresponding displacements (y). These p-y curves of liquefied soils are used to model the non-linear soil response of applied load vs. displacement. Rollins et al. (2005) has shown that p-y curves of liquefied soils have the following characteristics:

1. p-y curves of liquefied soils are characterized by a concave–up shape load-displacement curve. This shape appears to be due to dilative behavior of the soil during shearing.

2. p-y curves of liquefied sand transition from a concave–up shape to a concave-down shape as pore water pressure (u) dissipates after full liquefaction (R_u = excess pore pressure ratio = u/σ'_v = 1.0).

3. p-y curves of liquefied sand stiffen with depth. Smaller displacements are required to develop significant resistance.

4. p-y curves of liquefied sand become progressively stiffer after liquefaction due to pore water pressure dissipation.

5. p-y curves of liquefied sand exhibit almost no lateral resistance (zone of no lateral resistance) followed by a stiffening response occurring after a certain relative displacement.

6. p-y curve zone of no lateral resistance is smaller for larger piles when compared to smaller piles.

The computer program, LPile Plus (Reese, et. al., 2004), is typically used to evaluate lateral loads on piles using p-y curves. The p-y curves for liquefied soils (Rollins et al., 2005) that are included in LPile Plus attempt to model the strain hardening behavior of liquefied soils, but also tends to predict a response that is too soft. Since the Rollins model (Liquefied Sand in LPile Plus) has several limitations and response is very soft, it shall not be used to develop p-y curves of liquefied soils.

The p-y curves for Sand-Like soils subject to cyclic liquefaction should be estimated by either of the following two options:

1. The method recommended by Brandenberg et. al. (2007B) to develop p-y curves for fully liquefied Sand-Like soils (Excess pore pressure ratio = $R_u = u/\sigma'_v \approx 1.0$) consists of using static p-y curves for sands with a p-multiplier (m_p). The p-multiplier (m_p) values developed by Brandenberg et. al. (2007B) are shown in Figure 14-17. The p-multiplier (m_p) should be selected using the thick red line shown in Figure 14-17 that is consistent with the range recommended by Brandenberg et. al. (2007B) of 0.05 for loose sand to 0.30 for dense sand. The p-multiplier (m_p) is selected based on corrected SPT blow counts ($(N_1)_{60CS} = N^*_{1,60,CS}$ for Figure 14-17 only) which is consistent with the effects of relative density on undrained shear strength of sand. When limited liquefaction occurs ($0.20 \leq R_u = u/\sigma'_v < 1.0$), the p-multiplier ($m'_p$) can be estimated by the following equation:

$$\boldsymbol{m'_p} = 1 - \left[\boldsymbol{R_u}\left(1 - \boldsymbol{m_p}\right)\right] \qquad \text{Equation 14-9}$$

2. Alternatively, the static p-y curves for sands may be used to develop p-y curves for fully liquefied soils ($R_u \approx 1.0$) by using the liquefaction shear strength ratio (τ_{rl}/σ'_{vo}) to compute a reduced soil friction angle (ϕ_{rl}). The liquefaction shear strength ratio (τ_{rl}/σ'_{vo}) can be estimated from Chapter 13. The reduced soil friction angle due to cyclic liquefaction (ϕ_{rl}) can be computed by the following equation:

$$\boldsymbol{\phi_{rl}} = \mathbf{tan}^{-1}\left(\frac{\boldsymbol{\tau_{rl}}}{\boldsymbol{\sigma'_{vo}}}\right) \qquad \text{Equation 14-10}$$

The reduced soil friction angle for limited liquefaction ($\phi_{rl\text{-lim}}$) can be computed by the following equation:

$$\phi_{rl-\lim} = \mathbf{tan^{-1}}\left[\left(\mathbf{1} - \boldsymbol{R_u}\right)\left(\frac{\tau_{rl}}{\sigma'_{vo}}\right)\right] = \mathbf{tan^{-1}}\left[\left(\mathbf{1} - \boldsymbol{R_u}\right)\mathbf{tan}\left(\phi_{rl}\right)\right] \qquad \text{Equation 14-11}$$

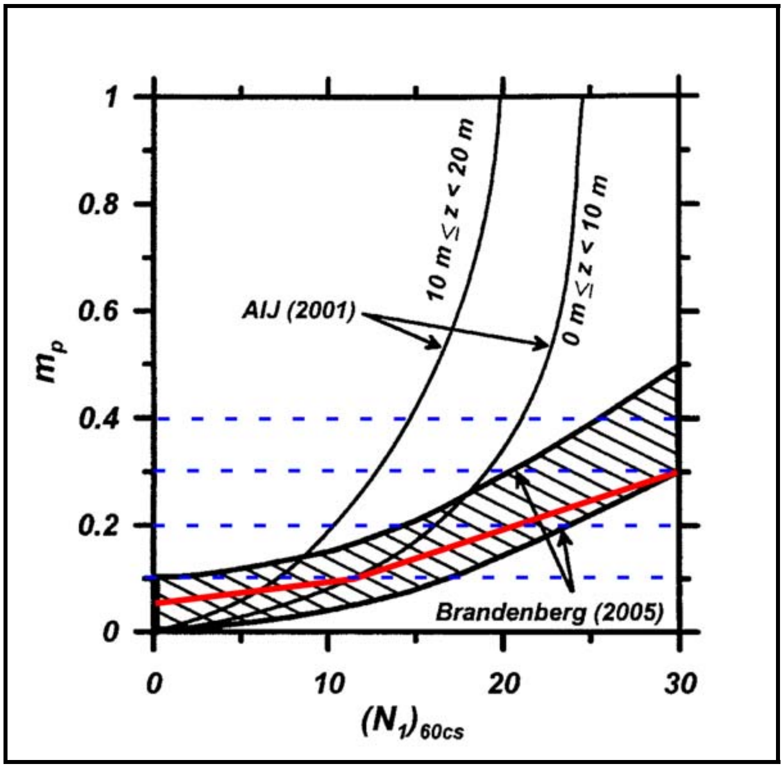


Figure 14-17, P-Multipliers (m_p) For Sand-Like Soils Subject to Cyclic Liquefaction (Modified Brandenberg, et. al., 2007B with permission from ASCE)

Lateral loadings of pile groups that are in fully liquefied soils (Excess pore pressure ratio = R_u = u/σ'_v > 1.0) are not influenced by p-multipliers when group effects are considered (even if closely spaced). During partial liquefaction ($0.20 \le R_u < 1.0$), prior to full liquefaction or after excess pore water pressures begin to dissipate, the group effects become increasingly apparent.

The preferred method to model lateral soil response is to use non-linear p-y curves. Because some structural software can only use linear springs to model lateral soil response, the secant modulus spring constant (K) can be computed for the corresponding displacement of a non-linear soil response model. The use of linear springs makes it necessary to adjust the linear spring constant (K) and it becomes an iterative process until displacements of the foundations match the displacement assumed in the development of the secant modulus spring constant.

14.8.4 Effects of Seismic Soil Instability On Deep Foundations

Seismic soil instability resulting from geotechnical seismic hazards can produce soil movements that can affect the performance of the bridge substructure (i.e. caps, single or group deep foundations, etc.). The soil-structure interaction can be very complex and will require a thorough understanding of the failure mechanism (Chapter 13). The proposed methodology for evaluating and modeling the soil-structure interaction is presented as guidance in evaluating the effects of seismic soil instability on deep foundations:

Step 1: Evaluate potential for instability due to geotechnical seismic hazard – Seismic hazard evaluation procedures are presented in Chapter 13.

Step 2: Evaluate potential for free field soil displacements – After the potential for instability has been confirmed, the maximum amount of the free-field displacements needs to be estimated. The preferred method for computing displacements is to use methods that are based on the sliding block displacement model proposed by Newmark (1965) that are presented in Chapter 13.

Step 3: Evaluate soil loadings and displacements of substructures – This consists of determining if soil flows around the substructure or if substructures are loaded, thereby, causing movement of the substructure. Evaluation procedures are presented in Section 14.8.5.

Step 4: Evaluate substructure performance and structural adequacy – The geotechnical engineer will need to provide the estimated displacements, loadings, and computed stresses (bending and shear) to the structural engineer for evaluation of the performance and structural adequacy of the substructures. If the substructures cannot be designed to meet the required performance and structural strength, there will be a need for mitigation as discussed in Step 5.

Step 5: Evaluate mitigation of geotechnical seismic hazard – The geotechnical engineer and the structural engineer will work together to evaluate the best mitigation strategy that will meet the EEI limit state performance objectives in the most safe and cost efficient manner as discussed in Section 14.15.

14.8.5 Evaluation of Soil Loading on Substructures

An evaluation of the soil loading on the substructures consists of evaluating which of the following two cases is occurring:

Case 1: The soil flows around the substructure with limited movement of the substructure. (See Figure 14-18)

Case 2: The soil and substructure move together. (See Figure 14-19)

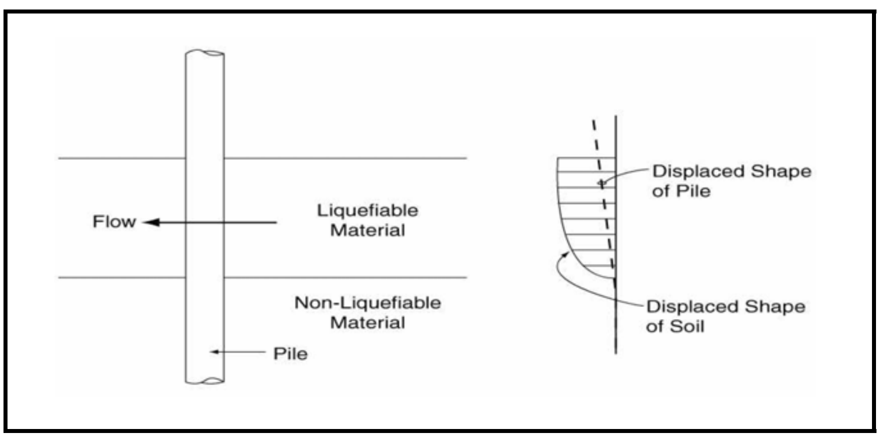


Figure 14-18, Flow of Soil Around Deep Foundation (NCHRP 12-49, 2001)

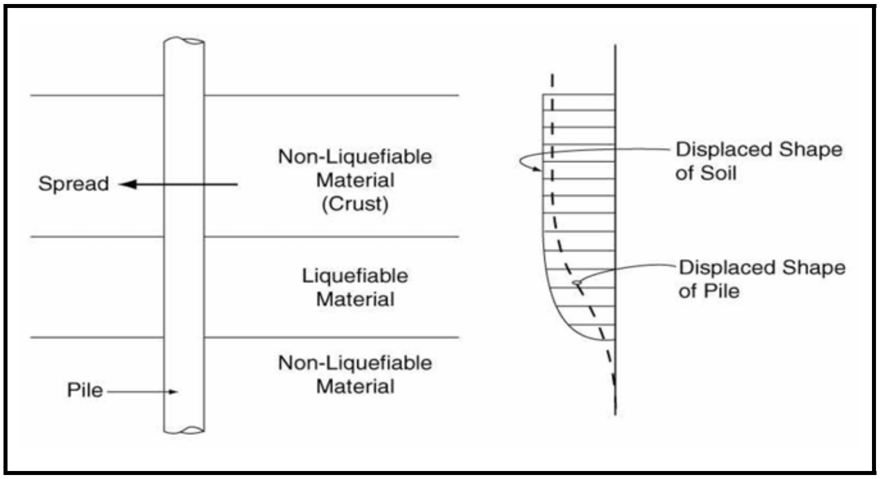


Figure 14-19, Movement of Soil and Deep Foundation (NCHRP 12-49, 2001)

The method to evaluate the loading Case on deep foundations is the displacement method presented in Section 14.8.7. If the free field soil displacements (Section 14.8.4, Step 2) are greater than deep foundation displacements, then the soil will be flowing around the substructure. When it is established that the soil will flow around the substructure, the substructure should be designed to withstand passive pressures computed by the force based methods presented in Sections 14.8.7.1 or 14.8.7.2.

If it is established that the substructure movements are similar in magnitude as the free field soil movements, the substructure will need to be evaluated for structural (i.e. yield strength) and performance (i.e. displacement, rotation, etc.) adequacy. Typically, when the soil does not flow around the substructure, the displacements of the substructure are relatively large and cause plastic deformations of the deep foundation and/or induce significant forces on the superstructure. When there is a crust of non-liquefied soils above the liquefied soils as shown in Figure 14-19, the passive forces tend to be significantly higher in the zone of non-liquefied material and the substructure will tend to move along with the free-field displacements. When a substructure includes a pile group within the crust of non-liquefied soils, the passive force load transfer can be evaluated as indicated in Section 14.8.7.

14.8.6 Soil Load Contribution on a Single Deep Foundation

The soil loading contribution from soil displacement on a single deep foundation (i.e. driven pile or drilled shaft) is very complex and generally depends on the soil shear strength, soil stiffness, spacing of piles, pile size (i.e. diameter or side dimension), arching effects, and construction method used to install the deep foundation. Although, there is no accepted method to evaluate the soil loading contribution, typical practice is that the contribution area of the soil loading or the effective pile width (B_{eff}) can be estimated as some multiple (λ) of pile size (B) where the soil loading contribution would be defined as (λB) for different soil shear strengths. It is accepted AASHTO LRFD practice that, if loading a single row of piles in the perpendicular direction and the pile spacing is less than 5B, there should be a reduction in resistance of the soil when using Beam on Nonlinear Winkler Foundation (BNWF) methods (i.e. Com624, LPile). Conversely, it can be estimated that the effective pile width ($B_{eff} = \lambda B$) on a single pile will range somewhere between one pile diameter up to 5 pile diameters ($B \leq \lambda B < 5B$). Until further research in this area becomes available, the effective pile width (B_{eff}) to be used for loading contribution shall be determined using the following equation:

$$B_{eff} = \lambda \cdot B \qquad \text{Equation 14-12}$$

Where,

B = Pile size (i.e. diameter or side dimension)

λ = effective pile width coefficient (Table 14-2)

ϕ = angle of internal friction of soil (backwall fill materials)

The effective pile width coefficient is determined as follows:

Table 14-2, Effective Pile Width Coefficient (λ)
(Adapted from NCHRP 12-70, 2008)

Pile Spacing	λ			
	Cohesive Soils		Cohesionless Soils	SSL of Sand-Like Soils
	Su ≤ 1000 psf & SSL of Clay-Like Soils	Su ≥ 1000 psf		
1B (side-by-side)	1	1	1	1
2B	1	2	$0.08\phi \le 3$	1
3B	1	2	3	1
>3B	1	2	3	1

14.8.7 Load Transfer Between Pile Group and Lateral Spreading Crust

Brandenberg et al. (2007A) proposed the Structural Model and the Lateral Spreading Model to evaluate the load transfer between pile groups and laterally spread crusts. In the Structural Model, the pile cap moves horizontally into a stationary soil mass. In the Lateral Spreading Model, the crust of non-liquefied soil moves laterally towards the stationary pile group. Brandenberg et al. (2007A) suggests that the actual loading condition would likely include some combination of ground displacement and pile cap displacement. The two load transfer models suggested by Brandenberg et al. (2007A) are to be used as an envelope of field loading behavior and do not capture the hysteretic dynamic behavior that actually occurs during shaking. A schematic to the pile group and block of stress influence in the non-liquefied crust is presented in Figure 14-20. A brief description of the Structural Model is presented in Section 14.8.7.1 and the Lateral Spreading Model discussed in Section 14.8.7.2. The geotechnical engineer is strongly encouraged to review and thoroughly understand the procedures presented in the Brandenberg et al. (2007A) original publication.

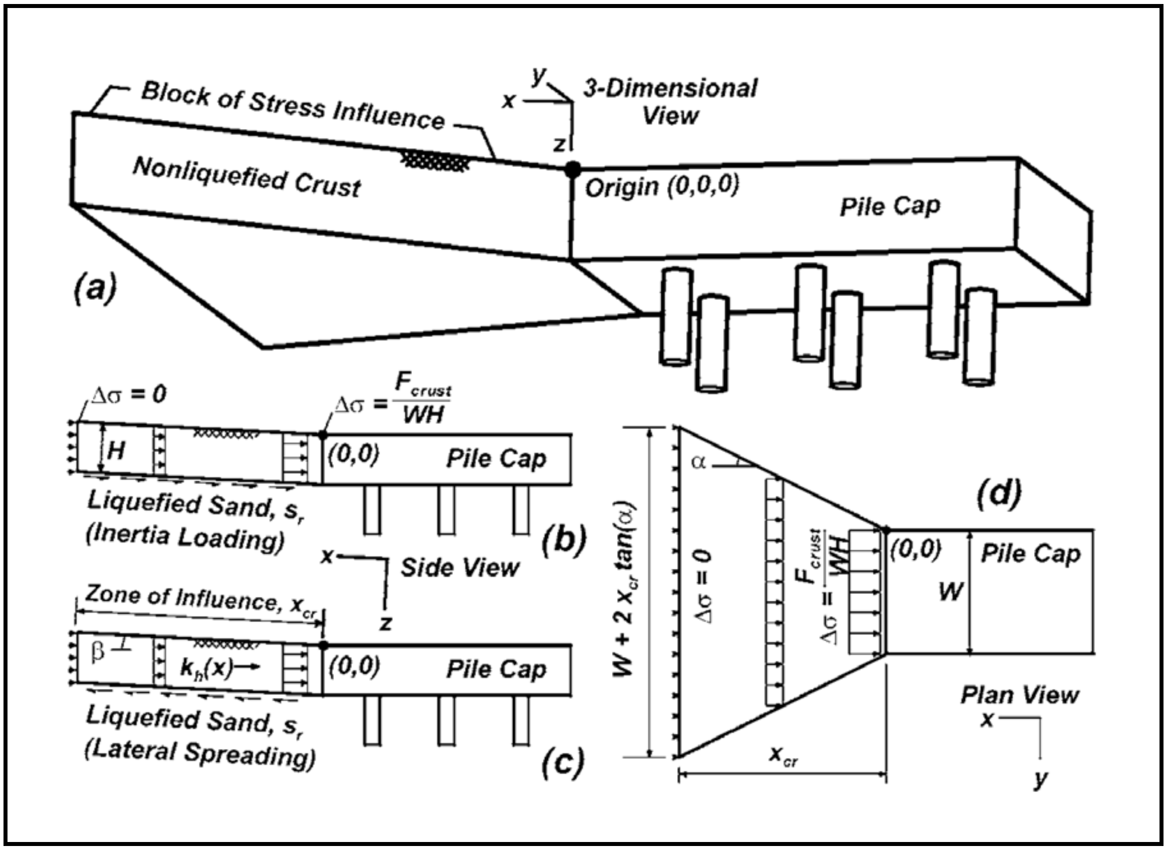


Figure 14-20, Load Transfer Between Pile Group and Lateral Spreading Crust (Brandenberg et al., 2007A with permission from ASCE)

14.8.7.1 Structural Model

The pile cap moves horizontally into a stationary soil mass in the Structural Model. This model represents the superstructure and/or pile cap inertia loading cycle and transient ground displacements are small. The assumptions of the Structural model are as follows:

1. Inertia of non-liquefied crust is neglected since the ground is assumed stationary.
2. The residual strength of the liquefied sand is fully mobilized along the base of the non-liquefied crust and acts in the downslope direction against the force imposed by the pile group (Figure 14-20(b)).
3. Stresses attenuate within a 3D block of stress influence that geometrically extends at an angle α from the backface of the pile cap in plan view (Figure 14-20(d)).

14.8.7.2 Lateral Spreading Model

The crust of non-liquefied soil moves laterally towards the stationary pile group in the Lateral Spreading Model. This may occur when the lateral spreading soil fails in passive pressure mode and flows around a laterally stiff pile foundation that exhibits little cap displacement. The assumptions of the Lateral Spreading model are as follows:

1. Pile cap displacement is zero.

2. The residual strength of the liquefied sand is fully mobilized along the base of the non-liquefied crust and acts upslope direction to resist lateral spreading of the crust (Figure 14-20(c)).

3. Stresses attenuate within a block of stress influence that geometrically extends at an angle α from the backface of the pile cap in plan view (Figure 14-20(d)).

4. Horizontal acceleration and downslope displacement at a given location in the nonliquefied crust layer must be compatible with the acceleration versus displacement relation obtained from sliding block solutions based on Newmark (1965) for a given ground motion.

14.8.8 Lateral Soil Loads Due to Seismic Hazard Displacements

Lateral loads on deep foundations resulting from seismic hazards can be evaluated by either displacement methods or force based methods. It may be necessary to use both methods to evaluate boundary conditions and reasonableness of the results.

Boulanger et al. (2003) and Brandenberg et al. (2007A, 2007B) have suggested modeling the effects of seismic hazard displacements using Beam on Nonlinear Winkler Foundation (BNWF) methods (i.e. Com624, LPile) that either use free-field soil displacement (Displacement Based Method: BNWF-SD) or limit pressures (Force Based Method: BNWF-LP) see Figures 14-21(a) and 14-21(b), respectively. The BNWF-SD method is a more general approach that consists of applying the Demand from seismic hazard free-field soil displacements (SD) on the free-end of the p-y soil springs. The BNWF-LP method consists of applying the limit pressures (LP) directly on the pile foundation and is therefore, a more restrictive approach, because it assumes that soil displacements are large enough to mobilize the ultimate loads from the spreading crust and liquefiable layer against the deep foundations. Application of the displacement boundary condition on the BNWF-SD method is typically more difficult than applying the force boundary condition on the BNWF-LP method. The BNWF-SD and the BNWF-LP methods are described in Sections 14.8.8.1 and 14.8.8.2, respectively. An alternate force based method that consists of using a limit equilibrium slope stability program is described in Section 14.8.8.3.

For either the displacement or force based method presented, the inertial forces should be included as static forces applied concurrently with the seismic hazard displacement demands.

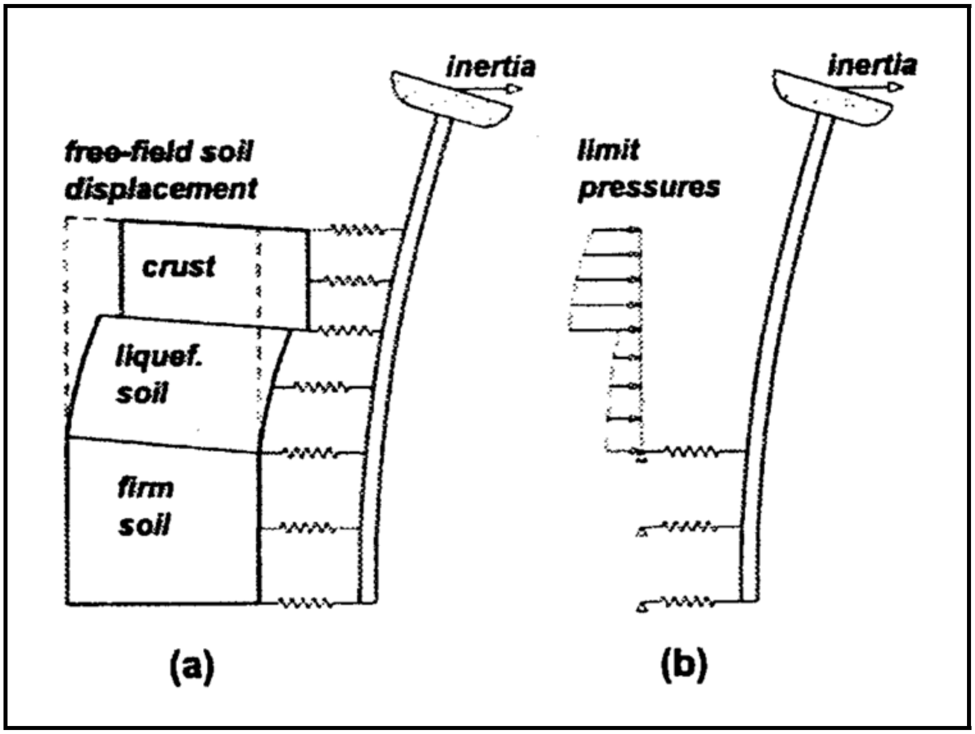


Figure 14-21, BNWF Methods For Evaluating Seismic Hazard Displacements (Brandenberg, et. al., 2007B with permission from ASCE)

14.8.8.1 Displacement Based Methods (BNWF-SD)

The displacement based method for evaluating the effects of lateral loads on deep foundations is based on procedures developed by Boulanger, et al. (2003) and Brandenberg, et al. (2007A, 2007B). This method uses LPile Plus or a similar computer program to perform the analysis. This performance based method is summarized below:

1. Estimate the free-field ground surface displacements of lateral spreading or slope instability in accordance with Chapter 13.

2. Estimate the lateral displacements as a function of depth. The shear strain profile approach described in Zhang et al. (2004) and illustrated by Idriss and Boulanger (2008) may be used. Another method would be to assume a constant displacement at the ground surface and crust and then vary displacements linearly with depth to the failure surface.

3. Model the free-field displacement and its displacement distribution with depth into LPile Plus computer program to compute the mobilized soil reaction vs. depth. Lateral resistance of liquefied soils (p_L) is modeled in accordance with Section 14.8.3. The lateral resistance (p_L) of liquefiable soils should be limited to:

$$p_L \leq 0.6\sigma'_{vo} b \qquad \text{Equation 14-13}$$

Where σ'_{vo} is the effective overburden stress before seismic loading and "b" is the pile width. The lateral load analysis using LPile Plus should consider the structural resistance of the foundation and the lateral resistance of the soil in front of the foundation as shown in Figure 14-22 (Imposed soil displacements).

4. Estimate the kinematic lateral loading effects on the deep foundation by using mobilized soil reaction vs. depth (Step 3) to evaluate deep foundation response using LPile Plus as shown in Figure 14-22 (Imposed pressure from spreading soil) to evaluate deflections, moments, shear, etc. Deep foundations should be evaluated for sufficient penetration below spreading soil to maintain lateral stability, location of plastic hinges, etc. If foundation resistance is greater than the applied pressures from the spreading soil, the soil will flow around the foundation. If the applied pressures from the spreading soil are greater than the deep foundation resistance, the foundation is likely to move along with the spreading soil. When pile caps are in contact with the spreading soil (or crust) passive pressures and side friction should be considered in the total loads applied to the foundation.

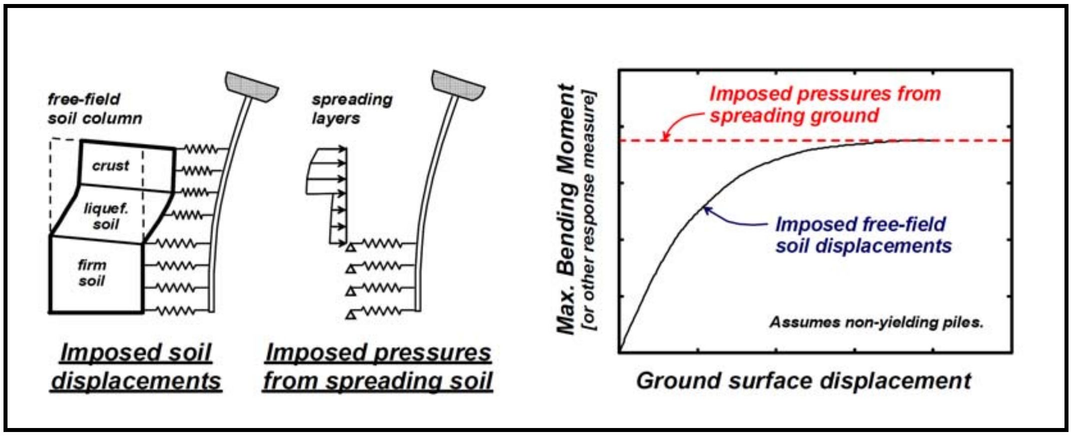


Figure 14-22, Methods for Imposing Kinematic Loads on Deep Foundations (Boulanger et al., 2003)

14.8.8.2 Force Based Methods (BNWF-LP)

This force based method for evaluating the effects of lateral loads on deep foundations is based on using limit pressures and BNWF modeling to evaluate lateral spreading induced loads on deep foundations. This method is based on back calculation from pile foundation failures caused by lateral spreading. Limit pressures assume that sufficient displacement occurs to fully mobilize lateral earth pressures. The limit pressure (LP) for non-liquefied soils is computed based on full passive pressures acting on the foundation. For liquefied soils, the limit pressure is computed as 30 percent of the total overburden pressure. The limit pressures are then imposed on the deep foundation using LPile Plus or a similar computer program to perform the analysis.

14.8.8.3 Force Based Methods (Slope Stability-LP)

This force based method also uses a limit equilibrium slope stability program to estimate the shear load the foundation must resist to achieve the target resistance factor, φ, (Chapter 9). The shear loads are then distributed as a limit uniform pressure within the liquefiable zone on the deep foundation using LPile Plus or similar computer program to perform the analysis.

14.9 BRIDGE ABUTMENT BACK WALL PASSIVE RESISTANCE

Earthquake-induced lateral loadings addressed in this Section are limited to those loadings that are a result of soil-structure interaction between soils and ERSs. Seismic lateral loadings can mobilize passive soil pressures such as those that occur at bridge abutments during a seismic event. Since the design methodologies presented in this Manual are performance-based, a nonlinear soil–abutment-bridge structure interaction model is used to compute the mobilized passive resistance as a function of displacement. The method to develop this model is based on the work by Shamsabadi (2006) and Shamsabadi et al. (2007). The basic framework of the model is a logarithmic spiral passive failure wedge coupled with a modified hyperbolic abutment-backfill stress-strain behavior (LSH). A hyperbolic force-displacement (HFD) curve is calculated by using the LSH relationship. The HFD curve is defined as shown in Figure 14-23.

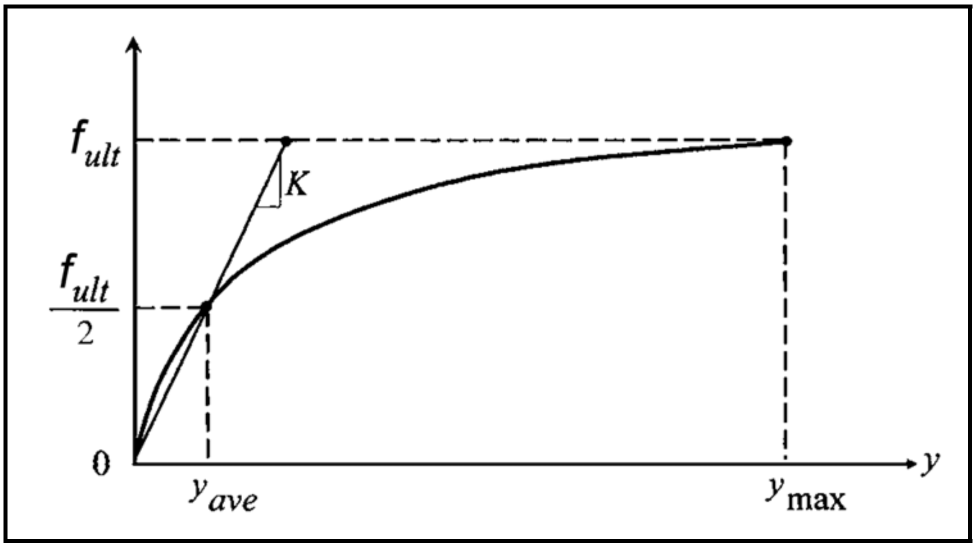


Figure 14-23, Hyperbolic Force-Displacement Formulation (modified Shamsabadi et al., 2007 with permission from ASCE)

Where F_{ult} is the maximum abutment force (kips) for the entire wall defined as indicated below:

$$F_{ult} = f_{ult} \cdot b_{wall} \qquad \text{Equation 14-14}$$

The maximum abutment force per unit width of wall is computed as follows:

$$f_{Ult} = p_{wall} \cdot h_{wall} \qquad \text{Equation 14-15}$$

Where,

f_{ult} = Maximum abutment force (kips per foot of wall width) developed at a maximum displacement (y_{max})

y_{max} = Maximum displacement (inches) where the maximum abutment force (f_{ult}) is developed

b_{wall} = Width of abutment wall (feet)

p_{wall} = maximum uniform wall pressure (ksf) developed at a maximum displacement (y_{max})

y_{max} = maximum displacement (inches) where the maximum abutment force (f_{ult}) is developed

h_{wall} = height of abutment wall (feet)

The maximum displacement where the maximum abutment force is developed is generally defined as y_{max}/h_{wall} =0.05 for Cohesionless soils and y_{max}/h_{wall} =0.10 for Cohesive soils. These values are in general agreement with those y_{max}/h_{wall} values observed by Clough and Duncan (1991) as shown in Table 14-3.

Table 14-3, Relative Movements Required To Reach Passive Earth Pressures (modified Clough and Duncan, 1991)

Type of Backfill	y_{max}/h_{wall}	y_{max}/h_{wall} [1]
Dense Sand	0.01	0.05
Medium Dense Sand	0.02	
Loose Sand	0.04	
Compacted Silt	0.02	
Compacted Lean Clay	0.05	0.10
Compacted Fat Clay	0.05	
Note: y_{max} = max movement at top of wall (feet); h_{wall} = height of wall (feet) [1] Recommended by Shamsabadi et al. (2007)		

The maximum uniform wall pressure p_{wall} has traditionally been 1.0 ksf for a 5.5 ft. high wall based on numerous full-scale tests for different types of backfill. The maximum wall pressure should be computed as indicated below:

$$p_{wall} = 0.5 \cdot K_{PE} \cdot \gamma_{Backfill} \cdot h_{wall} \qquad \text{Equation 14-16}$$

Where,

K_{PE} = Seismic passive earth pressure coefficient determined in accordance with Section 14.5.

$\gamma_{Backfilll}$ = Backfill unit weight (pcf)

h_{wall} = height of abutment wall (feet)

The average soil stiffness, K_{avg}, of the HFD curve is typically assumed to be 50 k/in/ft for cohesionless (granular) soils and 25 k/in/ft for cohesive soils. The average soil stiffness, K_{avg}, is defined as indicated below:

$$K_{avg} = \frac{0.5 f_{ult}}{y_{avg}} \qquad \text{Equation 14-17}$$

Where,

f_{ult} = maximum abutment force (kips per foot of wall width) developed at a maximum displacement (y_{max})

y_{avg} = average displacement (inches) where the maximum abutment force ($0.5F_{ult}$) is developed

The development of the HFD curve is based on the following three boundary conditions:

- Condition I: $f=0$ at $y_i=0$
- Condition II: $f=0.5F_{ult}$ at $y_i=y_{ave}$
- Condition III: $f=f_{ult}$ at $y_i=y_{max}$

The force-relationship in the general hyperbolic form is defined in the following equation:

$$f(y_i) = \frac{y_i}{A + By_i} \quad \text{Equation 14-18}$$

Where,

y_i = displacement

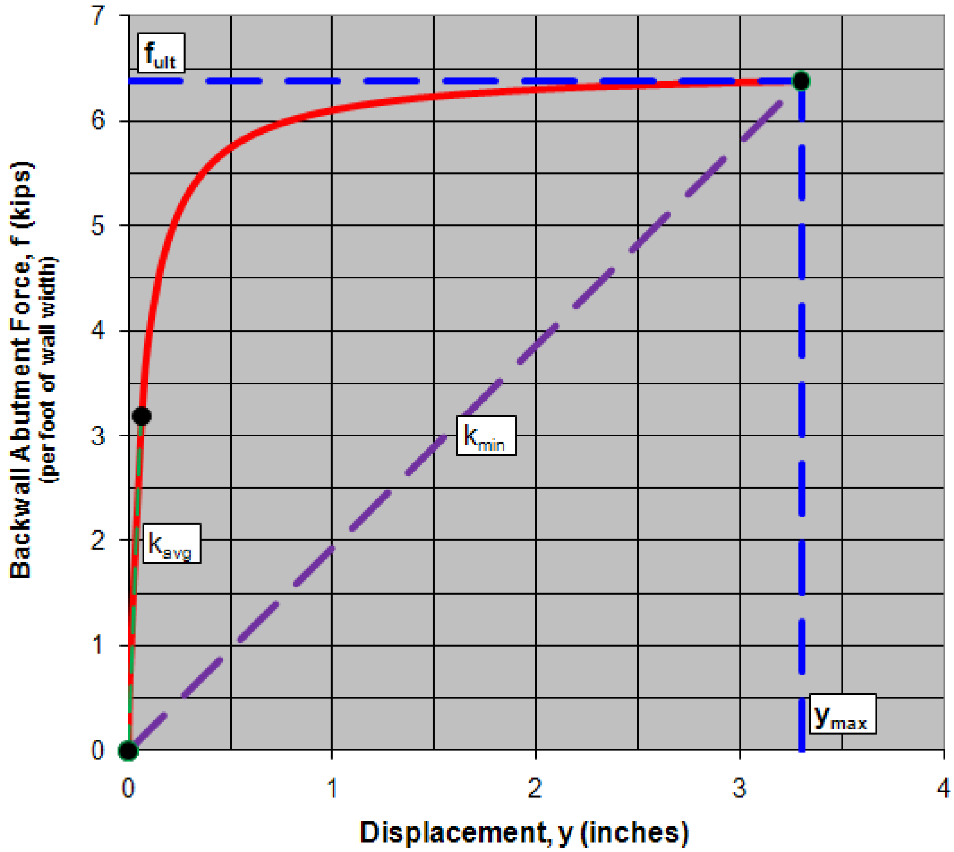
A are B are constants

$$A = \frac{y_{\max}}{2K_{avg}y_{\max} - f_{ult}} \quad \text{Equation 14-19}$$

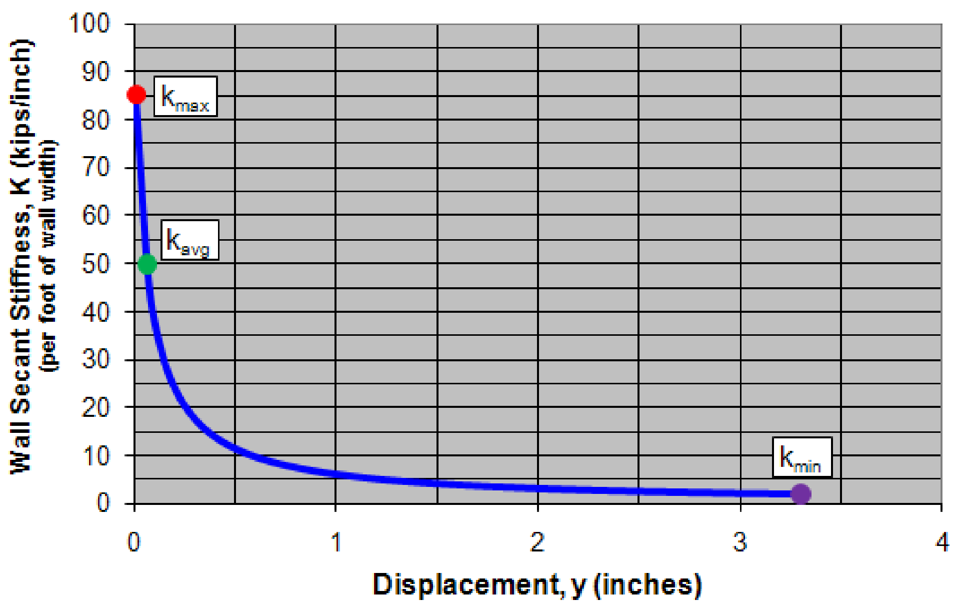
$$B = \frac{2(K_{avg}y_{\max} - f_{ult})}{f_{ult}(2K_{avg}y_{\max} - f_{ult})} \quad \text{Equation 14-20}$$

The geotechnical engineer shall provide the bridge designer appropriate hyperbolic force-displacement (HFD) curves (f vs. y) and wall secant modulus stiffness-displacement curves (K vs. y) similar to those shown in Figure 14-24 for Cohesionless Soils or Figure 14-25 for Cohesive Soils. The seismic passive pressures for cohesionless (granular) soil in Figure 14-24 is based on ϕ' = 28º; $\gamma_{Backfill}$ = 120 pcf; k_h = 0.23g; y_{max}/h_{wall} = 0.05; h_{wall} = 5.5 ft; p_{wall} = 1.16 ksf; K_{avg}=50 k/in/ft. The seismic passive pressures for cohesive soil in Figures 14-25 are based on c' = 50 psf; ϕ' = 26; $\gamma_{Backfill}$ = 115 pcf; k_h = 0.23g; y_{max}/h_{wall} = 0.10; h_{wall} = 5.5 ft; p_{wall} = 1.08 ksf; K_{avg}=25 k/in/ft.

Use K_{avg}=50k/in/ft and K_{avg}=25k/in/ft as default values for cohesionless and cohesive soils, respectively, unless site specific soils information is available. The shear strength parameters used to develop Figures 14-24 and 14-25 are long term (effective) stress parameters. Backwall design shall use long term (effective) stress parameters. Actual shear strength parameters used to design the embankments shall be used. If no shear strength data is required, the structure is on the same horizontal and vertical alignment, the shear strength parameters used to develop Figures 14-24 and 14-25 shall be used and are consistent with shear strength parameters for compacted soils as contained in Chapter 7.

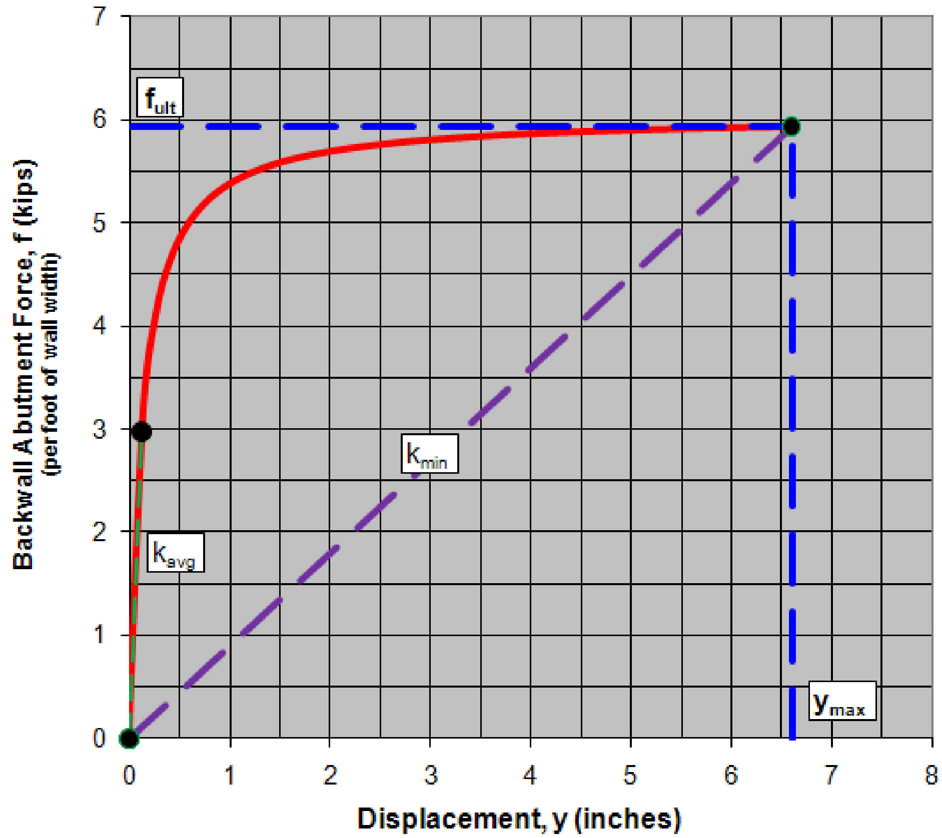


Hyperbolic Force-Displacement Curve

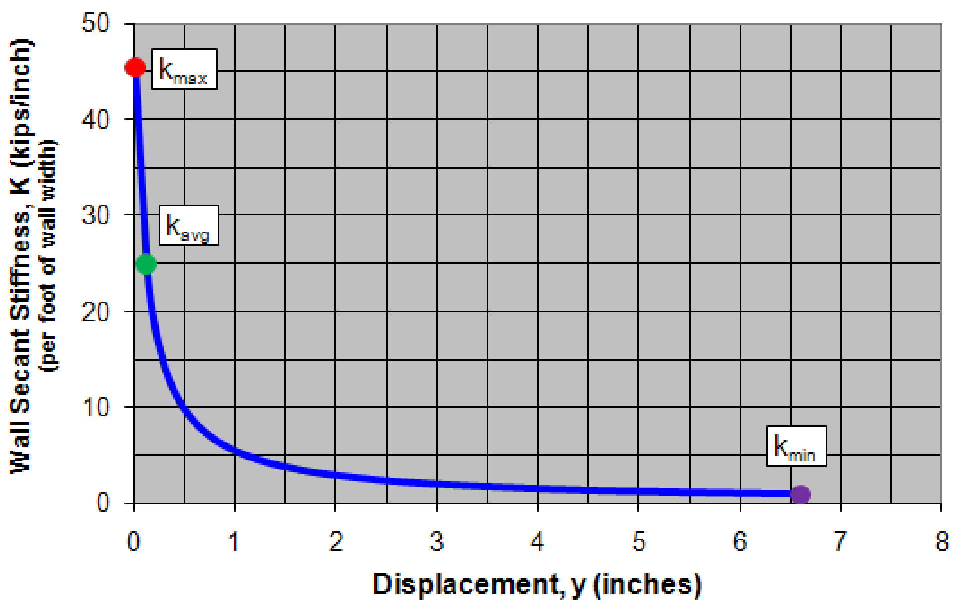


Wall Secant Stiffness-Displacement Curve

Figure 14-24, Bridge Abutment Backwall Passive Pressure - Cohesionless Backfill



Hyperbolic Force-Displacement Curve



Wall Secant Stiffness-Displacement Curve

Figure 14-25, Bridge Abutment Backwall Passive Pressure - Cohesive Backfill

The total seismic passive pressure ($F = F_{Passive}$) is dependent on the limits of the seismic passive wedge resistance that is located behind the back of the abutment wall as shown in

Figure 14-26. The limit of the seismic passive resistance wedge perpendicular to the back of the abutment wall is the distance "AE" as shown in Figure 14-9 and as shown in Figure 14-26. The limit along the wall is the effective width, B_{eff}, along the width of the abutment wall where the full seismic passive wedge (AE) can be fully included along the wall width. When the abutment is perpendicular to the longitudinal alignment of the bridge (skew angle, α = 0 deg.), the effective width (B_{eff}) is equal to the width of the abutment wall ($B_{Abutment}$). When bridge abutments are skewed, the geotechnical engineer will need to carefully evaluate the limits of the seismic passive wedge resistance. The effective width (B_{eff}) for skewed bridge abutments will typically be less than the width of the abutment wall ($B_{Abutment}$).

The abutment seismic passive pressures computed in this manner will provide resistance and displacements perpendicular to the back of the abutment wall. The resultant of the seismic demand acting perpendicular to the abutment wall face should be used to evaluate displacements (perpendicular to the back of the wall) resulting from seismic passive resistance.

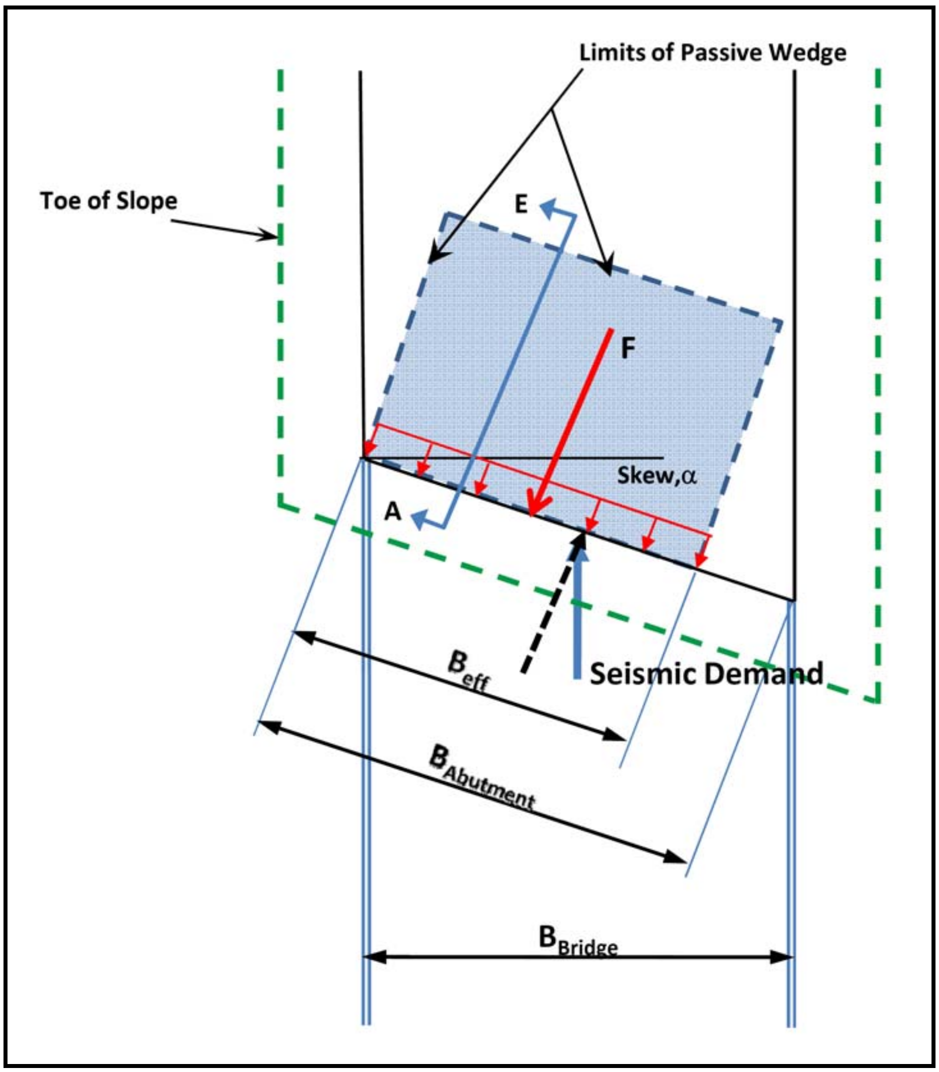


Figure 14-26, Skewed Bridge Abutment Backwall Seismic Passive Resistance

14.10 GEOTECHNICAL SEISMIC DESIGN OF EMBANKMENTS

As discussed in Chapter 17, slopes comprise two basic categories: natural and man-made (engineered). Typically man-made slopes are comprised of relatively uniform imported soils, thus allowing for more predictable performance and analysis during a seismic event. The design of these slopes becomes problematic when the embankment is placed on soft cohesive soils or loose cohesionless soils that can undergo Shear Strength Loss (SSL) as described in Chapter 13. Natural slopes present greater difficulties than man-made slopes, because of the potentially wide variation in soil type and shear strength that may be present in these slopes. As indicated in Chapter 17, natural slopes are those slopes that are formed from natural processes. Natural slopes are harder to analyze given the potential variability of not only material type and shear strength, but also thickness as well. Low shear strength layers that are thin may not be indicated in the geotechnical exploration and therefore could have potential consequences in not only seismic design, but also static design. Natural slopes are also hard to investigate because the terrain of most natural slopes is steep (1H:1V) making access extremely difficult. Natural slopes tend to fail during seismic events more frequently than man-made slopes. If the potential for SSL is present beneath the slope, the procedures discussed in Chapter 13 shall be followed.

The seismic design of the embankments shall conform to the procedures discussed in Chapter 13. Settlement induced by the seismic event for both unsaturated and saturated conditions shall be determined using the procedures discussed in Chapter 13. If seismic instability is determined to occur, all displacements determined shall conform to the performance limits discussed in Chapter 10. For displacements that exceed the limits of Chapter 10, see Section 14.15 for mitigation methods.

14.11 RIGID GRAVITY EARTH RETAINING STRUCTURE DESIGN

Rigid gravity earth retaining systems are comprised of gravity, semi-gravity and modular gravity ERSs as defined in Chapter 18 of this Manual (see Table 18-2). Gravity retaining structures use the weight of mass concrete and foundation soil to resist the driving forces placed on the structure. A rigid gravity earth wall is analyzed using the pseudo-static method as shown in Figure 14-1. Discussed in the following paragraphs are the requirements for determining the external stability of a rigid gravity ERS during a seismic event (Extreme Event I, EEI). The seismic internal stability calculations shall conform to the requirements contained in the AASHTO LRFD Bridge Design Specifications, except all accelerations used shall conform to the requirements of this Manual (i.e. k_{avg}). Additionally, all load and resistance factors shall conform to Chapters 8 and 9 and all displacements shall conform to Chapter 10. The external stability shall be determined using the following procedure:

Step 1: The first step in designing a rigid gravity ERS is to establish the initial ERS design using the procedures indicated in Chapter 18. This establishes the dimensions and weights of the rigid gravity ERS.

Step 2: Determine the PGA and S_{D1} using the procedures outlined in Chapter 12 regardless of whether the three-point method or a Site-Specific Seismic Response Analysis is performed. All ERSs are required to be designed for both EEI events (FEE and SEE).

Step 3: Determine the Peak Ground Velocity (PGV or V_{Peak}) using the correlation provided in Chapter 12.

Step 4: Compute the average seismic horizontal acceleration coefficient ($k_h = k_{avg}$) due to wave scattering as indicated in Chapter 13.

Step 5: Determine K_{AE} in accordance with the procedures described in Section 14.4.

Step 6: Compute the seismic active earth pressure force (P_{AE}), horizontal inertial force of the structural soil wedge (P_{IR}), and the horizontal inertial force of the slope surcharge above the structural soil wedge (P_{IS}) if the wall has a sloped backfill. The seismic force diagrams and appropriate variables are shown in Figure 14-27 for the level backfill surface case and Figure 14-28 for the sloped backfill surface case. For broken back backfill surface case, convert to a sloped backfill surface case using an effective backfill β angle in accordance with AASHTO LRFD Section 11 and then evaluate as shown in Figure 14-28. The width of the structural soil wedge (B_{SSW}) is the distance from the back of the wall to the heel of the footing as shown in Figures 14-27 and 14-28. The height (H_2) is the height where the seismic earth pressures are exerted on the structural wedge as is computed using the following equations:

Level Backfill Case (Figure 14-27):

$$\boldsymbol{H}_2 = \boldsymbol{H}_{Wall} + \boldsymbol{H}_{Ftg} \qquad \text{Equation 14-21}$$

Sloped Backfill Case (Figure 14-28):

$$\boldsymbol{H}_2 = \boldsymbol{H}_{Wall} + \boldsymbol{H}_{Ftg} + \boldsymbol{B}_{SSW} \tan(\beta) \qquad \text{Equation 14-22}$$

Where,

H_{Wall} = Height of wall stem
H_{Ftg} = Height of wall footing

Once the effective wall height (H_2) and the width of the structural soil wedge (B_{SSW}) have been determined, compute the seismic active earth force (P_{AE}) that is distributed as a uniform pressure over a height equal to H_2, the horizontal inertial force of the structural soil wedge (P_{IR}), and the horizontal inertial force of the soil surcharge above the structural soil wedge (P_{IS}) assumed to be triangular as shown in Figure 14-28. The horizontal seismic forces are computed as indicated below:

Level Backfill Case (Figure 14-27):

$$\boldsymbol{P}_{AE} = \gamma_P 0.5 \boldsymbol{K}_{AE} \gamma_{Backfill} (\boldsymbol{H}_2)^2 \qquad \text{Equation 14-23}$$

$$\boldsymbol{P}_{IR} = \gamma_P \boldsymbol{k}_{Avg} \boldsymbol{B}_{SSW} \boldsymbol{H}_{Wall} \gamma_{Backfill} \qquad \text{Equation 14-24}$$

Where,

γ_P = Permanent load factor, use 1.0 for ERS in EEI
K_{AE} = Seismic Active Earth Pressure Coefficient from Section 14.4

$\gamma_{Backfill}$ = Backfill Wet Unit Weight (pcf)
H_2 = Height where the seismic earth pressures are exerted on the structural wedge (feet) from Equation 14-21 (feet)
k_{Avg} = Average horizontal acceleration that accounts for wave scattering in accordance with Chapter 13.
B_{SSW} = Width of the structural soil wedge (feet)
H_{Wall} = Height of wall stem

Sloped Backfill Case (Figure 14-28):

$$\boldsymbol{P_{AE}} = \gamma_P 0.5 \boldsymbol{K_{AE}} \gamma_{Backfill} (\boldsymbol{H}_2)^2 \qquad \text{Equation 14-25}$$

$$\boldsymbol{P_{IR}} = \gamma_P \boldsymbol{k_{Avg}} \boldsymbol{B_{SSW}} \boldsymbol{H_{Wall}} \gamma_{Backfill} \qquad \text{Equation 14-26}$$

$$\boldsymbol{P_{IS}} = \gamma_P \boldsymbol{k_{Avg}} \mathbf{0.5} (\boldsymbol{B_{SSW}})^{\mathbf{2}} \mathbf{tan}(\beta) \gamma_{Backfill} \qquad \text{Equation 14-27}$$

Where,

γ_P = Permanent load factor, use 1.0 for ERS in EEI
K_{AE} = Seismic Active Earth Pressure Coefficient from Section 14.4
$\gamma_{Backfill}$ = Backfill Wet Unit Weight (pcf)
H_2 = Height where the seismic earth pressures are exerted on the structural wedge (feet) from Equation 14-22 (feet)
K_{Avg} = Average horizontal acceleration that accounts for wave scattering in accordance with Chapter 13.
B_{SSW} = Width of the structural soil wedge (feet)
H_{Wall} = Height of wall stem
β = Backslope angle (degrees)

Compute the horizontal inertial force of the weight of wall stem (F_{IW}) and horizontal inertial force of the weight of the wall footing (F_{IF}), as indicated by the following equations:

$$\boldsymbol{F_{IW}} = \boldsymbol{W_{Stem}} \boldsymbol{k_{Avg}} \qquad \text{Equation 14-28}$$

$$\boldsymbol{F_{IF}} = \boldsymbol{W_{Ftg}} \boldsymbol{k_{Avg}} \qquad \text{Equation 14-29}$$

Where,

W_{Stem} = Weight of wall stem
W_{Ftg} = Weight of wall footing
k_{Avg} = Average horizontal acceleration that accounts for wave scattering in accordance with Chapter 13.

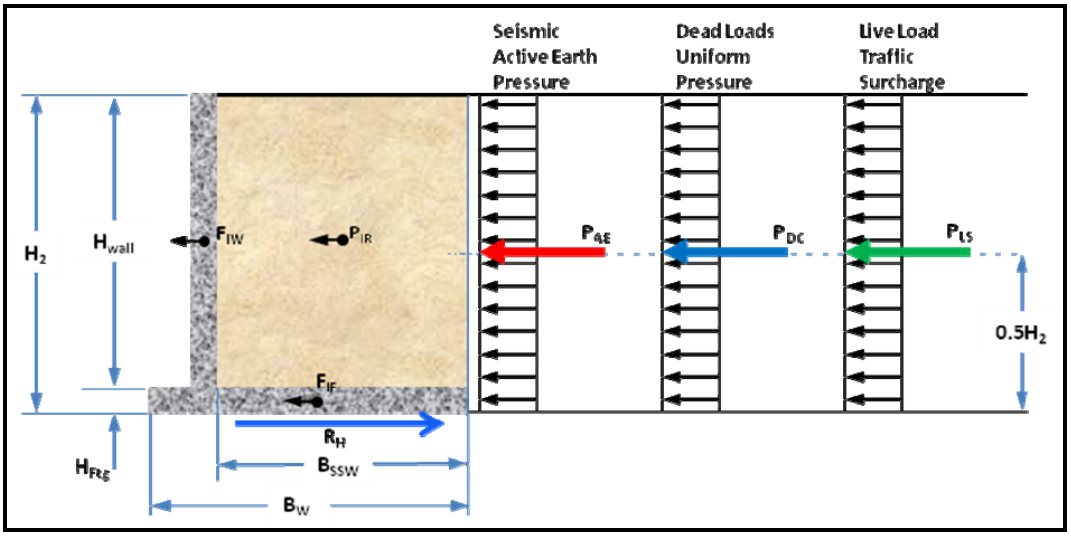


Figure 14-27, Rigid Gravity ERS Seismic Force Diagram – Level Backfill

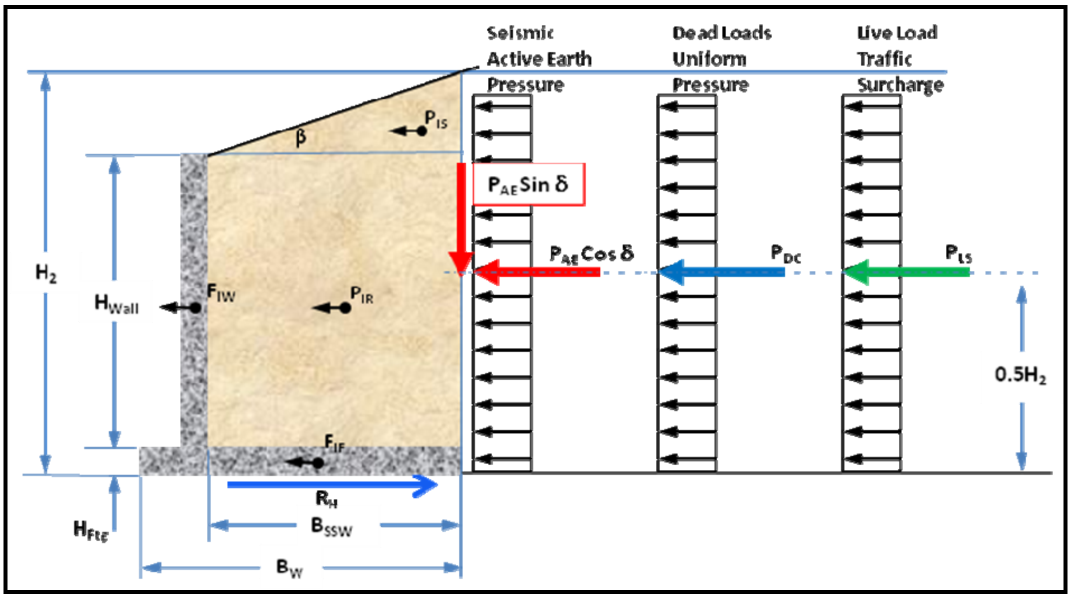


Figure 14-28, Rigid Gravity ERS Seismic Force Diagram – Sloping Backfill

Step 7: Compute dead load uniform pressure (P_{DC}), and live load traffic surcharge (P_{LS}) as indicated below:

$$P_{DC} = \gamma_P K_{AE} q_{DC} H_2 \qquad \text{Equation 14-30}$$

$$P_{LS} = \gamma_{EQ} K_{AE} q_{LS} H_2 \qquad \text{Equation 14-31}$$

Where,

γ_P = Permanent load factor, use 1.0 for ERS in EEI
K_{AE} = Seismic Active Earth Pressure Coefficient from Section 14.4
q_{DC} = Dead load uniform pressure (psf)

H_2 = Height where the seismic earth pressures are exerted on the structural wedge (feet). For level backfill surface use Equation 14-21 and for sloped backfill surface use Equation 14-22

γ_{EQ} = EEI load factor, See Chapter 8

q_{LS} = Live load traffic surcharge pressure (psf)

Step 8: Compute the horizontal driving forces (F_H) and the horizontal resisting forces (R_H). The passive earth pressure force shall only be included for shear keys that are located below the footing (see Section 14.5). The horizontal driving force (F_H) is determined by the following equation:

$$F_H = P_{AE}\cos\delta + P_{IR} + P_{IS} + P_{DC} + P_{LS} + F_{IW} + F_{IF} \quad \text{Equation 14-32}$$

Step 9: The frictional resistance of the foundation soils (R_{HF}) is computed using the following equations:

$$R_{HF} = \varphi\,\tau\,B_W = \varphi(c + N\tan(\delta_F))B_W \quad \text{Equation 14-33}$$

Where,

φ = Resistance factor, See Chapter 9

τ = Foundation soil shear strength

B_W = Base width of the wall footing (feet)

c = Cohesion of foundation soils

δ_F = Foundation-soil interface friction angle (AASHTO LRFD)

$\tan(\delta_F)$ = Foundation-soil interface friction coefficient, μ (AASHTO LRFD)

N = Normal force that is the sum of the vertical loads over the base width (B_W) of the reinforced soil mass. The normal force is computed by the following equation:

Equation 14-34

$$N = (B_{SSW}H_{Wall}\gamma_{Backfill}) + P_{AE}\sin\delta + (0.5B_{SSW}(H_2 - H_{Wall} - H_{Ftg})\gamma_{Backfill}) + W_W + W_F$$

Where,

B_{SSW} = Width of the structural soil wedge (feet)

H_{Wall} = Height of wall stem (feet)

$\gamma_{Backfill}$ = Backfill Wet Unit Weight (pcf)

P_{AE} = Seismic active earth force (lbs)

δ = 0° - level backfill or

β - sloping backfill

γ_P = Permanent load factor, use 1.0 for ERS in EEI

K_{AE} = Seismic Active Earth Pressure Coefficient from Section 14.4

H_2 = Height where the seismic earth pressures are exerted on the structural wedge (feet) from Equation 14-22 (feet)

K_{Avg} = Average horizontal acceleration that accounts for wave scattering in accordance with Chapter 13
B_{SSW} = Width of the structural soil wedge (feet)
β = Backslope angle (degrees)
W_{Stem} = Weight of wall stem
W_{Ftg} = Weigth of wall footing

Evaluate the ERS Wall bearing pressure, limiting eccentricity for overturning, sliding and global stability for the maximum seismic design load in accordance with the appropriate Chapters of the GDM. For determination of bearing capacity, see Chapter 15. If deep foundations are to be used to support flexible gravity ERSs, please contact the PCS/GDS. It should be noted that all EEI resistance factors (φ) are provided in Chapter 9. If all of the resistance factors are met, the static design is satisfactory and the seismic design is complete. It is reasonable to assume that if the demand/capacity ratio (D/C) for flexible gravity ERS meets the required resistance factors for the SEE design earthquake, the required resistance factors for the FEE design earthquake will be met. If bearing pressure and limiting eccentricity for overturning are not met, then the rigid gravity wall requires redesign to meet these resistances. If the sliding or global stability resistance factor criterion is not met, the ERS is unstable; therefore, continue to Step 10 and evaluate the displacements caused by both the FEE and SEE design earthquakes.

Step 10: Determine the yield acceleration (k_y) for the global stability of the ERS in accordance with Chapter 13. The k_y is the acceleration at which the ERS becomes just stable (i.e. φ = 1.0 = 1/FS = 1.0).

For evaluating the yield acceleration (k_y) for sliding of the ERS, first determine the driving forces (F_H) as a function of horizontal acceleration ($k = k_h$) and the resisting forces (R_H) as a function of horizontal acceleration ($k = k_h$) as depicted in Figure 14-29. The passive earth pressure force shall only be included for shear keys that are located below the footing (see Section 14.5); otherwise, no passive earth pressure force shall be used to resist lateral seismic forces. The yield acceleration (k_y) will be the location where the driving forces (F_H) and resisting forces (R_H) are equal.

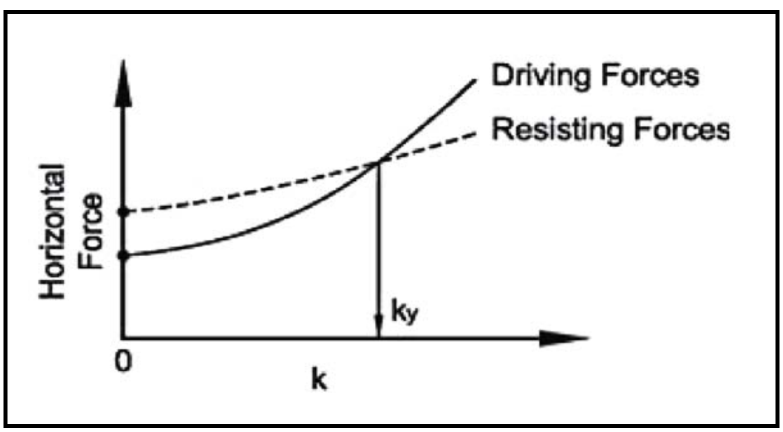


Figure 14-29, Determination of Yield Acceleration (k_y) (Anderson, et. al., 2008)

After determining k_y, determine the amount of displacement (d) using the procedures in Chapter 13. If the displacement is within the performance limits, as indicated in Chapter 10, then design is complete. If the displacement exceeds the performance limits in Chapter 10, redesign the ERS to achieve acceptable performance limits or determine if the amount of anticipated movement is acceptable to both the design team as well as SCDOT.

14.12 FLEXIBLE GRAVITY EARTH RETAINING STRUCTURE DESIGN

Flexible gravity earth retaining systems are comprised of gabion and Mechanically Stabilized Earth (MSE) ERSs as defined in Chapter 18 of this Manual (see Table 18-2). Flexible gravity retaining structures use the reinforced soil mass for MSE walls and the stone for gabion ERSs, and the foundation soils to resist the driving forces placed on the structure. Discussed in the following paragraphs are the requirements for determining the external stability of a flexible gravity ERS during a seismic event (Extreme Event I, EEI). The seismic internal stability calculations shall conform to the requirements contained in the AASHTO LRFD Bridge Design Specifications (Section 11.10 – Mechanically Stabilized Earth Walls), except all accelerations used shall conform to the requirements of this Manual (i.e. $A_s = k_{avg}$ as determined in Chapter 13). Additionally, all load and resistance factors shall conform to Chapters 8 and 9 and all displacements shall conform to Chapter 10. The external stability shall be determined using the following procedure:

Step 1: The first step in designing a flexible gravity ERS is to establish the initial ERS design using the procedures indicated in Chapter 18 and Appendix C (MSE walls). This establishes the dimensions and weights of the flexible gravity ERS.

Step 2: Determine the PGA and S_{D1} using the procedures outlined in Chapter 12 regardless of whether the three-point method or a Site-Specific Seismic Response Analysis is performed. All ERSs are required to be designed for both EEI events (FEE and SEE).

Step 3: Determine the Peak Ground Velocity (PGV = V_{Peak}) using the correlation provided in Chapter 12.

Step 4: Compute the average seismic horizontal acceleration coefficient ($k_h = k_{avg}$) due to wave scattering as indicated in Chapter 13.

Step 5: Determine K_{AE} in accordance with the procedures described in Section 14.4.

Step 6: Compute the seismic active earth pressure force (P_{AE}), the horizontal inertial force of the reinforced soil mass (P_{IR}) and the horizontal inertial force of the slope surcharge above the reinforced soil mass (P_{IS}) if wall has a sloped backfill. The seismic force diagrams and appropriate variables are shown in Figure14-30 for the level backfill surface case and Figure 14-31 for the sloped backfill surface case. For broken back backfill surface case, convert to sloped backfill surface case using an effective backfill β angle in accordance with AASHTO LRFD Section 11 and then evaluate as shown in Figure 14-31. The height (H_2) is the height where the seismic earth pressures and effective inertial wall width ($B_{Inertial}$) are computed as indicated below:

Level Backfill Case (Figure 14-30):

$$H_2 = H_{Wall} \qquad \text{Equation 14-35}$$

$$B_{Inertial} = \omega H_2 \qquad \text{Equation 14-36}$$

Where,

H_{Wall} = Height of MSE wall facing

ω = Coefficient equal to 0.70

Sloped Backfill Case (Figure 14-31):

$$H_2 = H_{Wall} + \left(\frac{\omega H_{Wall} \tan(\beta)}{1 - \omega \tan(\beta)} \right) \qquad \text{Equation 14-37}$$

$$B_{Inertial} = \omega H_2 \qquad \text{Equation 14-38}$$

Where,

H_{Wall} = Height of MSE wall facing (feet)

β = Backslope angle

ω = coefficient equal to 0.7 provided that $\omega H_{Wall} \leq B_W$

If $\omega H_{Wall} \geq B_W$ then,

$$\omega = \frac{B_W}{H_{Wall}} \qquad \text{Equation 14-39}$$

Where B_W is the base width of the wall as determined in Step 1. For MSE walls with concrete panel facing, the base width of the wall is taken from the back of the wall facing and for MSE walls with concrete block facing, the base width of the wall is taken from the front of the block facing.

Once the effective wall height (H_2) and the $B_{Inertial}$ variables have been determined, compute the seismic active pressures (P_{AE}). The seismic active earth pressure is distributed as a uniform pressure over a height equal to H_2. The horizontal inertial force of the reinforced soil mass (P_{IR}) and the horizontal inertial force of soil surcharge above the reinforced soil mass (P_{IS}), assumed to be triangular as shown in Figure 14-31, as indicated below:

Level Backfill Case (Figure 14-30):

$$P_{AE} = \gamma_P \, 0.5 K_{AE} \gamma_{Backfill} (H_2)^2 = \gamma_P \, 0.5 K_{AE} \gamma_{Backfill} (H_{Wall})^2 \qquad \text{Equation 14-40}$$

$$P_{IR} = \gamma_P k_{Avg} B_{Inertial} H_2 \gamma_R = \gamma_P k_{Avg} B_{Inertial} H_{Wall} \gamma_R \qquad \text{Equation 14-41}$$

Where,

γ_P	=	Permanent load factor, use 1.0 for ERS in EEI
K_{AE}	=	Seismic Active Earth Pressure Coefficient from Section 14.4
$\gamma_{Backfill}$	=	Backfill Wet Unit Weight (pcf)
γ_R	=	Reinforced Fill Wet Unit Weight (pcf)
H_2	=	Equal to the wall height, H_{Wall} (feet)
k_{Avg}	=	Average horizontal acceleration that accounts for wave scattering in accordance with Chapter 13.
$B_{Inertial}$	=	Effective inertial wall width from Equations 14-36 (feet)
β	=	Backslope angle (degrees)

Sloped Backfill Case (Figure 14-31):

$$P_{AE} = \gamma_P 0.5 K_{AE} \gamma_{Backfill} (H_2)^2 \qquad \text{Equation 14-42}$$

$$P_{IR} = \gamma_P k_{Avg} B_{Inertial} H_2 \gamma_R \qquad \text{Equation 14-43}$$

$$P_{IS} = \gamma_P k_{Avg} 0.5 (B_{Inertial})^2 \tan(\beta) \gamma_{Backfill} \qquad \text{Equation 14-44}$$

Where,

γ_P	=	Permanent load factor, use 1.0 for ERS in EEI
K_{AE}	=	Seismic Active Earth Pressure Coefficient from Section 14.4
$\gamma_{Backfill}$	=	Backfill Wet Unit Weight (pcf)
γ_R	=	Reinforced Fill Wet Unit Weight (pcf)
H_2	=	Equal to the wall height, H_{Wall} from Equation 14-37 (feet)
k_{Avg}	=	Average horizontal acceleration that accounts for wave scattering in accordance with Chapter 13.
$B_{Inertial}$	=	Effective inertial wall width from Equation 14-38 (feet)
β	=	Backslope angle (degrees)

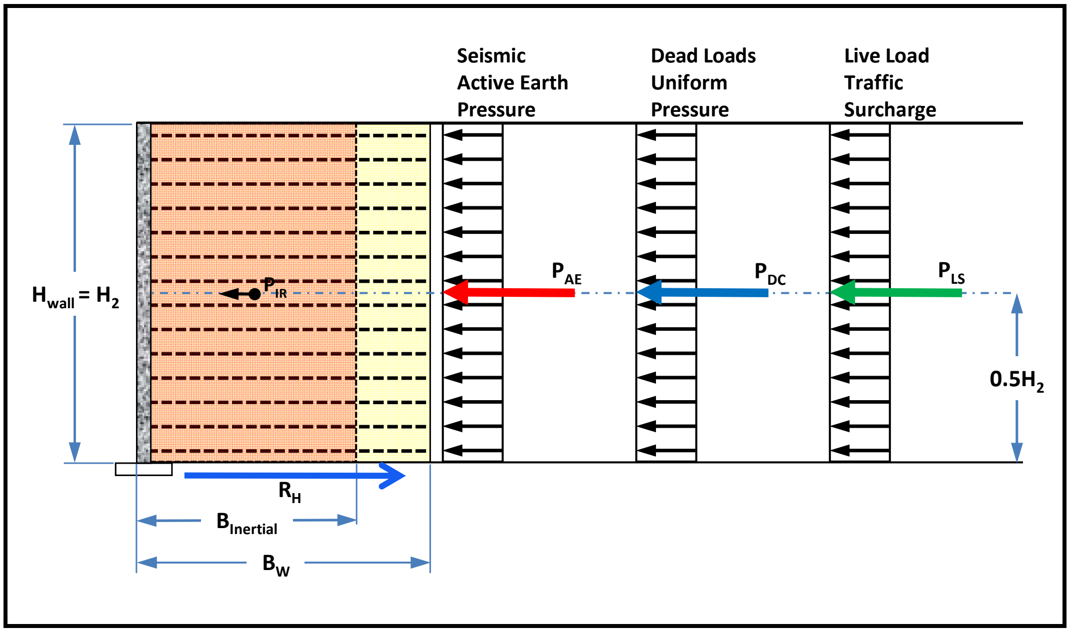


Figure 14-30, Flexible Gravity ERS Seismic Force Diagram – Level Backfill

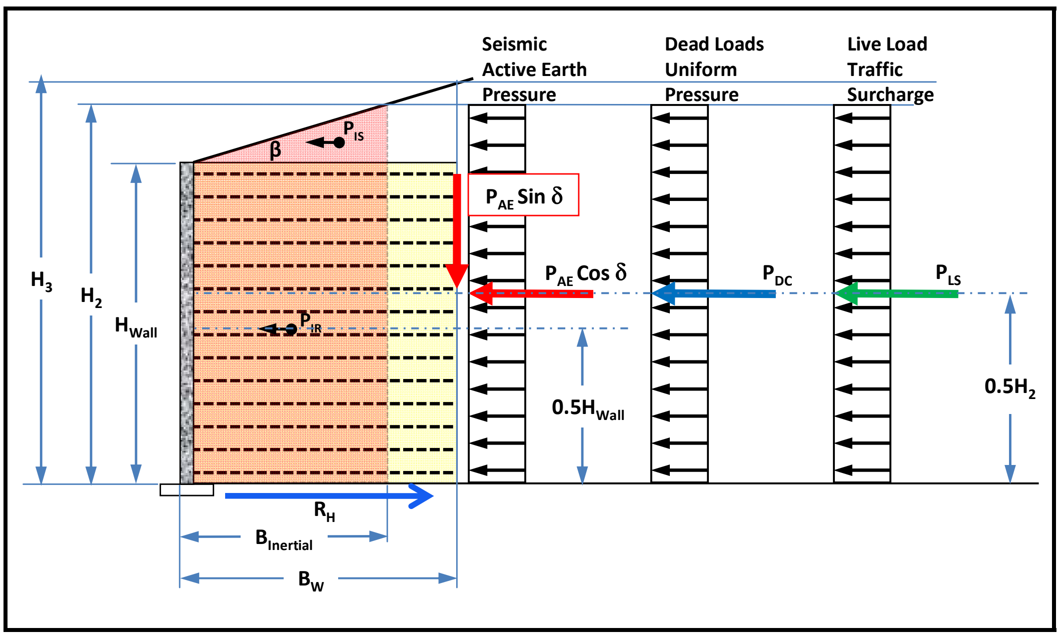


Figure 14-31, Flexible Gravity ERS Seismic Force Diagram – Sloping Backfill

Step 7: Compute dead load uniform pressure (P_{DC}), and live load traffic surcharge (P_{LS}) as indicated below:

$$P_{DC} = \gamma_P K_{AE} q_{DC} H_2 \qquad \text{Equation 14-45}$$

$$P_{LS} = \gamma_{EQ} K_{AE} q_{LS} H_2 \qquad \text{Equation 14-46}$$

Where,

γ_P = Permanent load factor, use 1.0 for ERS in EEI

γ_{EQ} = EEI load factor, See Chapter 8

K_{Avg}	=	Seismic Active Earth Pressure Coefficient from Section 14.4
q_{DC}	=	Dead load uniform pressure (psf)
q_{LS}	=	Live load traffic surcharge pressure (psf)
H_2	=	Equal to the wall height, H_{Wall} from Equation 14-37 (feet)
K_{Avg}	=	Average horizontal acceleration that accounts for wave scattering in accordance with Chapter 13.
$B_{Inertial}$	=	Effective inertial wall width from Equation 14-38 (feet)

Step 8: Compute the horizontal driving forces (F_H) and the horizontal resisting forces (R_H). No passive earth pressure force shall be used to resist horizontal driving forces on MSE walls. The horizontal driving force (F_H) is determined by the following equation:

$$F_H = P_{AE}\cos\delta + P_{IR} + P_{IS} + P_{DC} + P_{LS} \qquad \text{Equation 14-47}$$

The resisting force (R_H) is determined using the lesser of soil-soil frictional resistance within the reinforced soil mass ($R_{HSoil\text{-}Soil}$), soil-reinforcement frictional resistance within the reinforced soil mass ($R_{HSoil\text{-}Reinf}$), or the frictional resistance in the foundation soils (R_{HF}).

$$R_H = R_{HSoil-Soil}, R_{HSoil-\mathrm{Reinf}}, \textit{or } R_{HF} \quad \textbf{(lesser of resisting forces)} \qquad \text{Equation 14-48}$$

Where the frictional resistance forces are computed using the following equations:

$$R_{HSoil-Soil} = \varphi N \tan(\phi_R) B_W \qquad \text{Equation 14-49}$$

$$R_{HSoil-\mathrm{Reinf}} = \varphi N \tan(\rho) B_W \qquad \text{Equation 14-50}$$

$$R_{HF} = \varphi \tau B_W = \varphi(c + N\tan(\phi_R))B_W \qquad \text{Equation 14-51}$$

Where,

φ	=	Resistance factor, See Chapter 9.
ϕ_R	=	Internal friction angle of the reinforced fill
ρ	=	Friction angle between the soil reinforcement and the reinforced fill material. For continuous reinforcement (sheet type) $\rho = 0.67\phi_R$. For all other non-continuous reinforcement (strip type) $\rho = \phi_R$.
B_W	=	Base width to the wall as defined in Step 6
δ	=	0° - level backfill or β - sloping backfill
N	=	Normal force that is the sum of the vertical loads over the base width (B_W) of the reinforced soil mass. The normal force is computed by the following equation:

$$N = (B_W H_{Wall} \gamma_R) + P_{AE} \sin\delta + (0.5B_W (H_3 - H_{Wall}) \gamma_{Backfill}) \qquad \text{Equation 14-52}$$

Step 9: Evaluate the MSE Wall bearing pressure, limiting eccentricity for overturning, sliding, and global stability for the maximum seismic design load in accordance with the appropriate Chapters of the GDM and Appendix C. For determination of bearing capacity see Chapter 15. If deep foundations are to be used to support flexible gravity ERSs, please contact the PCS/GDS. It should be noted that all EEI resistance factors (φ) are provided in Chapter 9. If all of the resistance factors are met, the static design is satisfactory and the seismic design is complete. It is reasonable to assume that if the demand/capacity ratio (D/C) for flexible gravity ERS meets the required resistance factors for the SEE design earthquake, the required resistance factors for the FEE design earthquake will be met. If bearing pressure and limiting eccentricity for overturning are not met, then MSE Wall requires redesign to meet these resistances. If the sliding or global stability resistance factor criterion is not met, the ERS is unstable; therefore, continue to Step 10 and evaluate the displacements caused by both the FEE and SEE design earthquakes.

Step 10: Determine the yield acceleration (k_y) for the global stability of the ERS in accordance with Chapter 13. The k_y is the acceleration at which the ERS becomes just stable (i.e. φ = 1.0 = 1/FS = 1.0).

For evaluating the yield acceleration (k_y) for sliding of the ERS, first determine the driving forces (F_H) as a function of horizontal acceleration ($k = k_h$) and the resisting forces (R_H) as a function of horizontal acceleration ($k = k_h$) as depicted in Figure 14-32. No passive earth pressure force shall be used to resist lateral seismic forces. The yield acceleration (k_y) will be the location where the driving forces (F_H) and resisting forces (R_H) are equal.

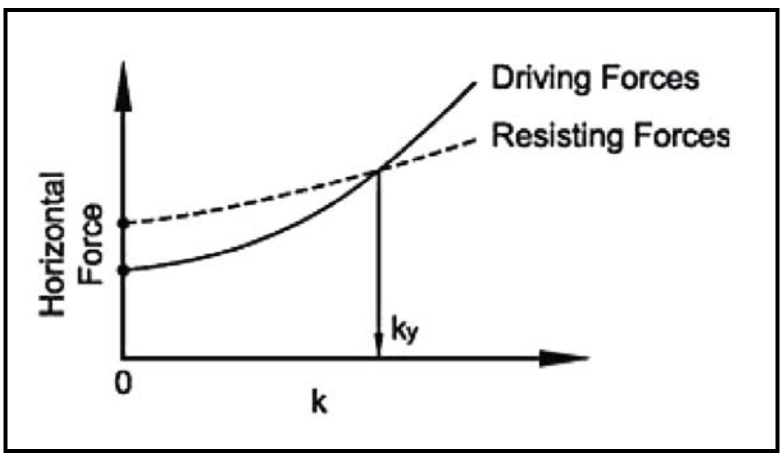


Figure 14-32, Determination of Yield Acceleration (k_y) (Anderson, et. al., 2008)

After determining k_y determine the amount of displacement (d) using the procedures in Chapter 13. If the displacement is within the performance limits as indicated in Chapter 10, then design is complete. If the displacement exceeds the performance limits in Chapter 10, redesign the ERS to achieve acceptable performance limits or determine if the amount of anticipated movement is acceptable to both the design team as well as SCDOT.

14.13 CANTILEVER EARTH RETAINING SYSTEM DESIGN

Cantilevered earth retaining systems are comprised of unanchored sheet-pile and soldier pile and lagging and anchored sheet-pile and soldier pile and lagging ERSs as defined in Chapter 18 of this Manual (see Table 18-1). Unanchored cantilevered walls will be discussed first and then anchored cantilevered walls.

14.13.1 Unanchored Cantilever ERSs

Design unanchored cantilever ERSs is to establish the initial ERS design using the procedures indicated in Chapter 18 of this Manual. This establishes the dimensions of the cantilever ERS. Typically unanchored cantilevered ERSs are limited to a height of less than 16 feet. It should be noted that it is customary to ignore the inertial loadings of the structure members.

Step 1: Determine the PGA and S_{D1} using the procedures outlined in Chapter 12 of this Manual regardless of whether the three-point method or a Site-Specific Seismic Response Analysis is performed. All ERSs are required to be designed for both EEI events (FEE and SEE). It is reasonable to assume that if an unanchored cantilever ERS satisfies the required resistance factors for the SEE, the required resistance factors for the FEE will be met.

Step 2: Determine the Peak Ground Velocity (PGV or V_{Peak}). Chapter 12 provides an equation for determining the PGV.

Step 3: Compute the average seismic horizontal acceleration coefficient ($k_h = k_{avg}$) due to wave scattering as indicated in Chapter 13.

Step 4: Determine K_{AE} and P_{AE} in accordance with the procedures described in Section 14.4 of this Chapter. The passive earth pressure coefficient K_{PE} and the passive earth pressure, P_{PE}, shall be determined as indicated in Section 14.5.

Step 5: Evaluate the structural requirements using either a suitable software package or by hand calculation (i.e. using something similar to the free earth support method). Confirm that the displacements predicted to achieve active earth pressure conform to the performance limits provided in Chapter 10.

Step 6: Check the global stability using the procedures outlined in Chapter 17. The acceleration used in the global stability analysis shall be the k_{avg} as determined in Step 4. If the resistance factor (φ) is greater than 1.0, determine displacements and compare to Chapter 10. If the displacements are within limits, design is complete. If the displacements exceed the limits, redesign the wall and begin again at Step 1.

14.13.2 Anchored Cantilever ERSs

Design anchored cantilever ERSs is to establish the initial ERS design using the procedures indicated in Chapter 18 of this Manual. This establishes the dimensions of the cantilever ERS. Typically anchored cantilevered ERSs have heights in excess of 15 feet, but typically no more than 70 feet. It should be noted that it is customary to ignore the inertial loadings of the structure members.

Step 1: Determine the PGA and S_{D1} using the procedures outlined in Chapter 12 of this Manual regardless of whether the three-point method or a Site-Specific Seismic Response Analysis is performed. All ERSs are required to be designed for both EEI events (FEE and SEE). It is reasonable to assume that if an anchored cantilever ERS satisfies the required resistance factors for the SEE, the required resistance factors for the FEE will be met.

Step 2: Determine the Peak Ground Velocity (PGV or V_{Peak}). Chapter 12 provides an equation for determining the PGV.

Step 3: Compute the average seismic horizontal acceleration coefficient ($k_h = k_{avg}$) due to wave scattering as indicated in Chapter 13.

Step 4: Determine K_{AE} and P_{AE} in accordance with the procedures described in Section 14.4 of this Chapter. The passive earth pressure coefficient K_{PE} and the passive earth pressure, P_{PE}, shall be determined as indicated in Section 14.5.

Step 5: Evaluate the structural requirements using the same pressure distribution from the static analysis. From the resulting loading diagram, check the loads on the tendons and grouted anchors to confirm that seismic loads do not exceed the loads applied during proof testing of each anchor. Confirm that the grouted anchors are located outside of the active seismic pressure failure wedge.

Step 6: Check the global stability using the procedures outlined in Chapter 17. The acceleration used in the global stability shall be the k_{avg} as determined in Step 4. If the resistance factor (φ) is greater than 1.0, determine displacements and compare to Chapter 10. If the displacements are within limits, design is complete. If the displacements exceed the limits, redesign the wall and begin again at Step 1.

14.14 IN-SITU REINFORCED RETAINING SYSTEM DESIGN

In-situ reinforced earth retaining systems are comprised of soil nail wall ERSs as defined in Chapter 18 of this Manual (see Table 18-1).

Design in-situ reinforced ERSs is to establish the initial ERS design using the procedures indicated in Chapter 18 of this Manual. This establishes the dimensions of the soil nail ERS. Typically soil nail ERSs have heights in excess of 10 feet but typically no more than 70 feet.

Step 1: Determine the PGA and S_{D1} using the procedures outlined in Chapter 12 of this Manual regardless of whether the three-point method or a Site-Specific Seismic Response Analysis is performed. All ERSs are required to be designed for both

EEI events (FEE and SEE). It is reasonable to assume that if a soil nail ERS satisfies the required resistance factors for the SEE, the required resistance factors for the FEE will be met.

Step 2: Determine the Peak Ground Velocity (PGV or V_{Peak}). Chapter 12 provides an equation for determining the PGV.

Step 3: Compute the average seismic horizontal acceleration coefficient ($k_h = k_{avg}$) due to wave scattering as indicated in Chapter 13.

Step 4: Use the k_{avg} determined previously in a pseudo-static analysis of the soil nail ERS, using a commercially available software (i.e. SNAIL® or GOLDNAIL®). If the resistance factor (φ) is less than 1.0, the design is acceptable. However, if the resistance factor (φ) is greater than 1.0, determine displacements and compare to Chapter 10. First, determine the yield acceleration (k_y) corresponding to the point where the horizontal driving and resisting forces are equal (i.e. φ = 1.0 or FS = 1.0). After determining k_y, determine the amount of displacement (d) using the procedures in Chapter 13. If the displacement is within the performance limits as indicated in Chapter 10, then design is complete. If the displacement exceeds the performance limits in Chapter 10 either redesign the ERS to achieve acceptable performance limits or determine if the amount of anticipated movement is acceptable to both the design team as well as SCDOT.

14.15 SEISMIC HAZARD MITIGATION

If the performance limits provided in Chapter 10 are exceeded, then the design team must decide if mitigation is practical or not (i.e. do nothing). “Doing nothing” is a decision that must be made by the entire design team. In some cases, this may be the most viable option. For instance, if flow failure is anticipated, the cost of mitigation would be excessively prohibitive; therefore, it would be less expensive to accept the movement. If mitigation is practical, there are two categories, either structural mitigation or geotechnical mitigation.

14.15.1 Structural Mitigation

Structural mitigation is where the structure is designed to handle the displacements anticipated during the seismic event. The decision to use structural mitigation should be made by the entire design team.

14.15.2 Geotechnical Mitigation

Geotechnical mitigation is where the soil beneath and around the structure is modified to prevent displacements occurring during a seismic event from exceeding the limits contained in Chapter 10. Additional guidance on geotechnical mitigation methods may be found in Idriss and Boulanger (2008). Geotechnical mitigation efforts are limited to those indicated in Chapter 19 or those approved by the PCS/GDS on a project specific basis.

14.15.3 Selection of Mitigation Method

The selection of the appropriate mitigation strategy should be based on the cost of the mitigation method, the anticipated results of the mitigation method and the amount of post-seismic displacement that is anticipated to occur. The need for mitigation should be identified as early as possible in the design process to allow for time to consider all mitigation alternatives.

14.16 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.

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Chapter 15

SHALLOW FOUNDATIONS

Final

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June 2010

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CHAPTER 15

SHALLOW FOUNDATIONS

15.1 INTRODUCTION

This Chapter presents the design and analysis requirements for shallow foundations that will be used for projects designed both within and for SCDOT. According to the *Bridge Design Manual* a shallow foundation "distributes the loads…to suitable soil strata or rock at relatively shallow depths (less than 10 feet)". Shallow foundations are used not only to support bridges, but also to support building structures, earth retaining structures (ERS) (see Chapter 18), box and floorless culverts and other ancillary structures. Shallow foundations are not limited to spread footings, but may also include strip footings, mat foundations and thickened (turned-down) edge slabs. The type of shallow foundation to be used will be based on the structure to be supported. The *Bridge Design Manual* lists the following prohibitions to the use of shallow foundations:

- at stream crossings where (shallow foundations) may be susceptible to scour,
- on fills, and
- beneath bents that are located within the reinforced soil mass associated with MSE walls.

In addition, shallow foundations shall not be used to support bridges or building structures if the potential for liquefaction is indicated, unless approved in writing by the PCS/GDS. For these structures the magnitude of displacement shall be determined using the procedures provided in Chapter 13 of this Manual and shall conform to the requirements of Chapter 10 of this Manual.

Geotechnical Engineering Circular No. 6 – Shallow Foundations indicates that a strip footing has a length dimension (L_f) at least five times larger than the width dimension (B_f). Spread footings have a ratio of L_f/B_f less than five. Mat foundations according to Bowles (1996) are very large spread footings that have thickenesses ranging from 2-1/2 to 6-1/2 feet and have negative moment steel. A thickened edge slab is a variation of a mat foundation, where the interior of the slab is typically thin, 4 to 6 inches in thickness, while at the locations of columns and at the edge the thickness is at least 18 inches.

15.2 DESIGN CONSIDERATIONS

The design of shallow foundations consists of two components, the bearing (resistance to shear) capacity and settlement (performance limits). According to Bowles (1996) most shallow foundation problems occur because of settlement, while true bearing failure is limited. Typically, the factored resistance (R_r) will be dictated by the settlement (performance limits, see Chapter 10). Roadway embankments do not typically have a structural foundation element; however, either settlement or global stability (Chapter 17) will govern the design and acceptability of the embankment. Therefore, it is not required or necessary to determine the bearing capacity of the soil beneath embankments, unless there is a question of localized shearing failure. Shallow foundations shall be designed for service (settlement), strength (bearing capacity) and extreme event (bearing capacity) loading states as required by LRFD. All shallow foundation designs will be governed by the basic LRFD equation:

$$\boldsymbol{Q} = \sum \eta_i \gamma_i \boldsymbol{Q_i} \le \varphi \boldsymbol{R_n} = \boldsymbol{R_r}$$ **Equation 15-1**

Where,

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

Shallow foundations shall be proportioned so that the factored resistance is not exceeded when loading is applied to the foundation and the performance limit of the foundation is not exceeded. Further, the affect of inclined loads that cause the reduction of the net bearing area shall also be considered. The bearing depth of shallow foundations depends on the type of structure being built. The bearing depths for shallow foundations are discussed in greater detail in the following sections.

15.2.1 Bearing Depth – Bridge Foundations

The bearing depth of shallow foundations used to support bridges shall be determined in accordance with the latest edition of the *Bridge Design Manual*.

15.2.2 Bearing Depth – Other Structures

The bearing depth of shallow foundations used to support structures (i.e. buildings, ERSs other than MSE walls, signs, etc.) shall account for the presence of groundwater and frost penetration. Shallow foundations should not be placed beneath the groundwater table since this will require additional effort in construction. To prevent frost from affecting shallow foundations, shallow foundations shall be placed beneath the frost penetration depth, which according to the Building Code Council for South Carolina is between 1 and 2 inches. The bottom of shallow foundations shall be placed no shallower than 18 inches unless the depth to the groundwater table is shallower than this depth. If the depth to the groundwater table is shallower than 12 inches, contact the PCS/GDS with recommendations for installing the shallow foundations prior to completing foundation design plans.

15.2.3 Bearing Depth – Embankments and MSE Walls

The bearing capacity for embankments and MSE walls shall be determined from the existing ground surface (i.e. d = 0). The leveling pad of an MSE wall is not a shallow foundation and does not have to meet the requirements of this chapter.

15.3 BEARING CAPACITY DETERMINATION

The nominal bearing capacity of a shallow foundation shall be determined using the Strength limit state in accordance with the procedures published in the latest version of the AASHTO

LRFD Bridge Design Specifications (Section 10.6 – Spread Footings). The size of the foundation shall be determined using the factored resistance. This proportionally sized foundation shall be used in the determination of settlement (Service limit state). In addition, the proportionally sized foundation shall also be used to check for sliding of the foundation due to inclined and shear loads. The nominal bearing capacity of foundations placed within slopes shall also be determined in accordance with the latest version of the AASHTO LRFD Bridge Design Specifications (Section 10.6 – Spread Footings). Further, the proportionally sized foundation shall be checked for the Extreme Event limit state. Both bearing and settlement shall be determined for the Extreme Event. The bearing determined for the Extreme Event shall be compared to and not exceed the nominal resistance. The settlement determined (Chapter 13) for the Extreme Event shall be compared to the performance limits provided in Chapter 10. The resistance factors provided in Chapter 9 are for shallow foundations with vertical loads. The effect of inclined loads on the resistance factor is not well known or understood; therefore caution should be used when applying the resistance factors of Chapter 9 to shallow foundations with inclined loads. The AASHTO LRFD Bridge Design Specifications allow for the use of plate load tests to determine the bearing capacity of soil; however, the use of plate load tests to determine bearing capacity is not allowed by SCDOT.

15.4 SETTLEMENT

As indicated previously, settlement normally governs the size and capacity for shallow foundations. The total settlement as well as the differential settlement (the difference in settlement between two points) should be considered when sizing a shallow foundation. Further, the time for settlement to occur as well as the rate of settlement (amount per unit of time) should also be considered in shallow foundation design. The amount and time for settlement to occur shall be determined using the methods described in Chapter 17. The amount (total and differential) of settlement and the rate of settlement shall conform to the limits presented in Chapter 10 of this Manual.

Typically for shallow foundations founded on cohesionless materials the amount of settlement will be relatively small and will typically occur during construction. For cohesive soils the amount of settlement can be quiet large and can take a long time to occur. Therefore, preloading may be used to reduce or remove the anticipated settlement amount prior to installation of the shallow foundations. If preloading is performed, the pressure applied by the preload should achieve at least one-half of the factored bearing resistance required. Under this condition additional settlement will occur and should be determined, as should the time for this settlement to occur. According to AASHTO LRFD Bridge Design Specifications three-dimensional affects should be considered if the following criteria is met.

$$\frac{B_f}{H_o} \leq 4 \qquad \textbf{Equation 15-2}$$

Where,

B_f = B = Foundation width

H_o = H = Total thickness of consolidating layer

Then the settlement should be reduced using the following equation

$$S_{c(3D)} = \lambda S_{c(1D)} \qquad \textbf{Equation 15-3}$$

Where,

$S_{c(1D)}$ = Total primary consolidation

λ = Three-dimensional reduction factor (see Figure 15-1)

$S_{c(3D)}$ = Reduced total primary consolidation accounting for three-dimensional effects

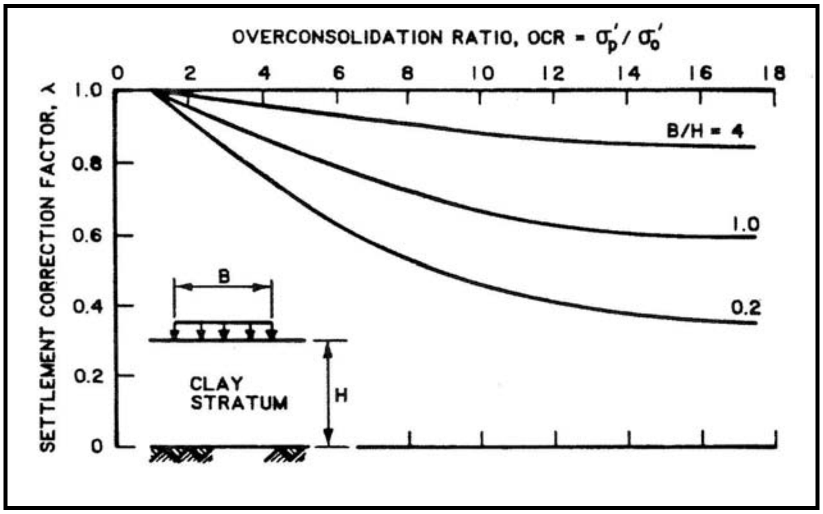

Figure 15-1, Three-Dimensional Reduction Factors (Settlement Analysis – 1990)

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Chapter 16

DEEP FOUNDATIONS

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CHAPTER 16

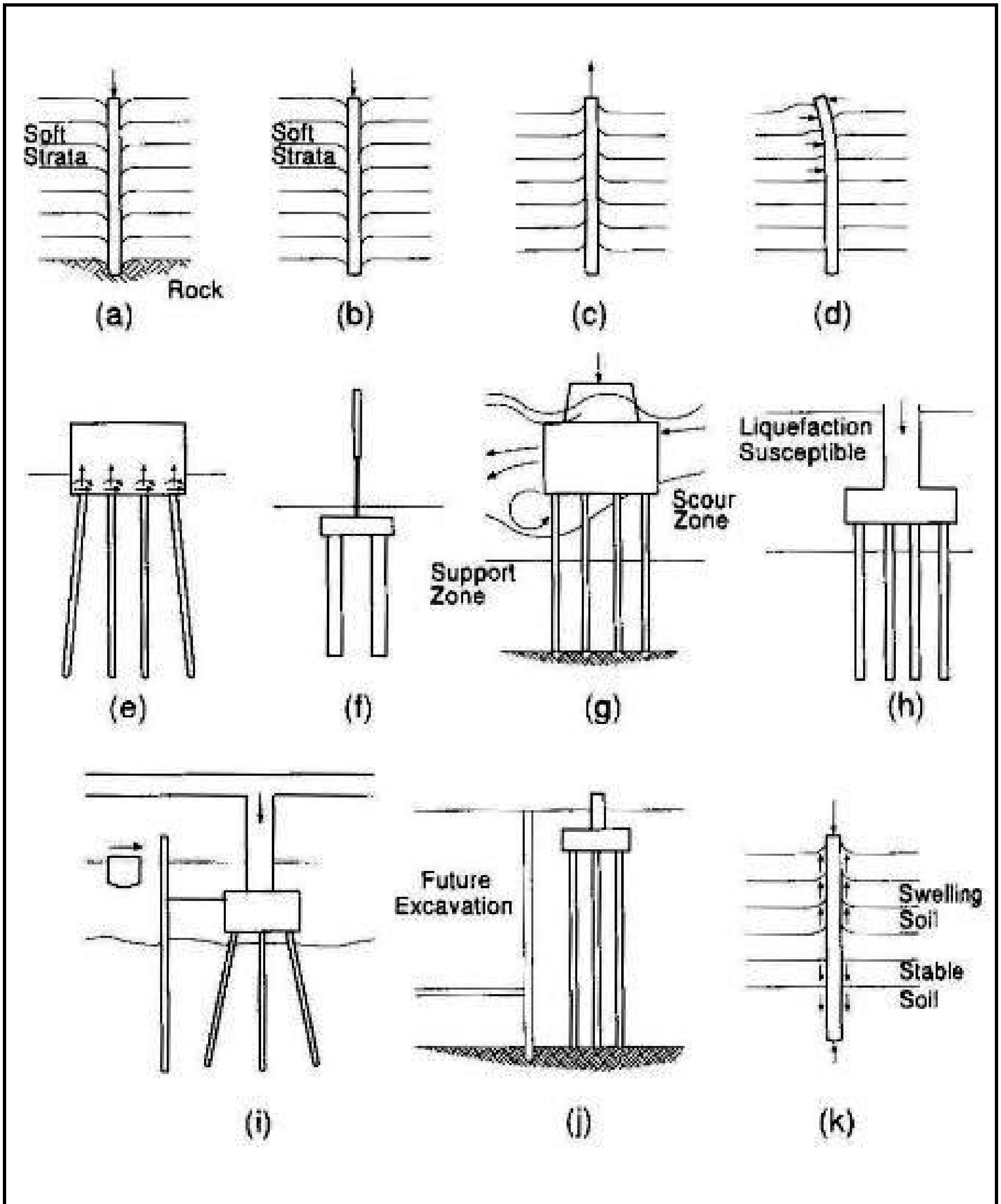
DEEP FOUNDATIONS

16.1 INTRODUCTION

This Chapter provides general guidance in the design and analysis of deep foundations used to support highway structures. Deep foundations are used in lieu of shallow foundations on the majority of SCDOT projects. Deep foundations consist of driven piles, drilled shafts or piers, drilled piles, auger cast-in-place (continuous flight auger, CFA) piles and micro-piles. Each foundation type has specific advantages and disadvantages that will be discussed in subsequent Sections. The design of deep foundations is comprised of 2 components, the bearing (resistance to shear) capacity and settlement (performance limits); however, in the design of deep foundations bearing typically governs.

According to NAVFAC DM-7.2 deep foundations are defined as developing bearing at depths (D_f) greater than 5 times the size (diameter) (B_f) of the foundation (i.e. $D_f \geq 5B_f$). As indicated previously, bearing typically governs the design of deep foundations not settlement. The bearing of deep foundations is based on either the end bearing (Q_t) or skin friction (S_f) along the shaft of the foundation acting independently of the other component or a combination of the two components acting together. Deep foundations need to be considered for several reasons:

- When the upper soil strata are too weak or compressible to support the required vertical loads (a), (b), (c) (letters refer to Figure 16-1);
- When shallow foundations cannot adequately support inclined, lateral, or uplift loads, and overturning moments (d), (e), (f);
- When scour around foundations could cause loss of bearing capacity at shallow depths (g);
- When soils around foundations are subjected to liquefaction during seismic events (h);
- When fender systems are required to protect bridge piers from vessel impact (i);
- When future excavations are planned which would require underpinning of shallow foundations (j), and;
- When expansive or collapsible soils are present, which could cause undesirable seasonal movements of the foundations (k).



Note: Illustrations (e) and (i) above shows battered piles, please note that SCODT prefers vertical piles.

Figure 16-1, Reasons for Deep Foundations (Hannigan et al, 2006)

All deep foundation designs will be governed by the basic LRFD equation.

$$Q = \sum \eta_i \gamma_i Q_i \leq \varphi R_n = R_r = \varphi R_t + \varphi R_s$$ Equation 16-1

Where,

Q = Factored Load
Q_i = Force Effect
η_i = Load modifier
γ_i = Load factor
R_r = Factored Resistance (i.e. allowable capacity)
R_n = Nominal Resistance (i.e. ultimate capacity)
R_t = Nominal Tip Resistance
R_S = Nominal Skin Friction Resistance
φ = Resistance Factor

Selection of the resistance factor (φ) will be discussed in greater detail in the following Sections. Typically, the resistance factor is based on the method of construction control for piles and on the type of material and where (i.e. end or side) the capacity is developed. SCDOT does not use design method specific resistance factors (see Chapter 9). The factored load is provided by the bridge (structural) engineer.

16.2 DESIGN CONSIDERATIONS

The design of deep foundations supporting bridge piers, abutments, or walls should consider all limit state loading conditions applicable to the structure being designed. A discussion of the load combination limit states that are used in deep foundation design is discussed in Chapter 8 and the Deep Foundation performance limit with corresponding limit state is reproduced below in Table 16-1. 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 16-1, Deep Foundation Limit States

Performance Limit	Limit States		
	Strength	**Service**	**Extreme Event**
Axial Compression Load	√		√
Axial Uplift Load	√		√
Structural Capacity[1]	√		√
Lateral Displacements		√	√
Settlement		√	√

[1]Determined by Bridge (Structural) Engineer

16.2.1 Axial Load

Axial loadings should include both compressive and uplift forces in evaluation of deep foundations. Forces generated from the Strength limit state and Extreme Event limit state are used to determine nominal axial pile resistances from the axial design process. 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. The Extreme Event limit states are design boundary conditions considered to represent an excessive or improbable loading combination. Such conditions may include ship impacts, vehicle impact, and seismic events. Because the probability of this event occurring during the life of the structure is relatively small, a smaller safety margin is appropriate when evaluating this limit state.

The static capacity of a pile/shaft can be defined as the sum of soil/rock resistances along the pile/shaft surface and at the pile/shaft toe available to support the imposed loads on the pile. A static analysis is performed to determine the nominal bearing resistance (R_n) of an individual pile/shaft and of a pile/shaft group as well as the deformation response of a pile and/or group to the applied loads. The nominal bearing resistance (R_n) of an individual pile and of a pile group is the smaller of:

(1) the capacity of surrounding soil/rock medium to support the loads transferred from the pile/shaft or,

(2) the structural capacity of the pile/shaft.

The static pile/shaft capacity from the sum of the soil/rock resistances along the pile/shaft surface and at the pile/shaft toe can be estimated from geotechnical engineering analysis using:

(1) Laboratory determined shear strength parameters of the soil and rock surrounding the pile;

(2) Standard Penetration Test (SPT) data;

(3) In-Situ Test data (i.e. CPT); or

(4) Full scale load test data.

16.2.2 Lateral Load

Lateral loadings applied in foundation design should consider foundation members placed through embankments, locations on, near or within a slope, loss of support due to erosion or scour, and the bearing strata significantly inclined. Forces generated from the Service limit state and Extreme Event limit state are used to determine the horizontal and vertical movements of the foundation system. The Service limit state is a design boundary condition for structure performance under intended service loads, and accounts for some acceptable measure of structure movement throughout its performance life. The Extreme Event limit states are design boundary conditions considered to represent an excessive or improbable loading combination. Such conditions may include ship impacts, vehicle impact, and seismic events. Because the probability of this event occurring during the life of the structure is relatively small, a smaller safety margin is appropriate when evaluating this limit state.

16.2.3 Settlement

The amount of settlement is normally limited to the amount required to develop the capacity of the deep foundation element. Settlements are determined for the Service limit state. The settlement is not normally determined on individual driven piles, micro-piles or CFAs, while the settlement on individual shafts is normally determined. For deep foundations, the settlement of group is normally determined. In addition, the elastic shortening of the deep foundation elements due to the load should be included in the overall settlement. The inclusion of elastic shortening is required, since the performance of the structure will be affected by this movement. Static analysis calculations of the deformation response to lateral loads and of pile/shaft groups' settlement are compared to the performance criteria established for the structure.

16.2.4 Scour

The design of deep foundations shall consider the effects of scour on the capacity and length requirements of the foundation. The nominal capacity of deep foundations shall be determined for the soils beneath the scourable soils. The depth of scour shall be determined by the Hydraulic Engineering Group. The capacity of the scourable soils shall be added to the nominal capacity of driven piles when developing driving criteria, but no such increase in capacity is required for the drilled shafts, CFAs and micro-piles because they are not driven. The following Sections will provide additional details for handling scour for each foundation type.

16.2.5 Downdrag

Downdrag on deep foundations is caused by two distinct phenomena, settlement of subgrade soils and seismically induced liquefaction. Settlement is normally anticipated to occur at the end bent of bridges where the bridge meets the road embankment. Downdrag induced settlements are applied to the Strength limit state of the deep foundation. Downdrag loads will be discussed in the following Sections, while the settlement of the embankments is discussed in Chapter 17. The other phenomenon that can cause downdrag is seismically induced liquefaction. This downdrag load is applied to the Extreme Event limit state and will be discussed in the following Sections in greater detail. The amount of seismically induced liquefaction settlement is determined using the procedures outlined in Chapter 13.

16.3 DRIVEN PILES

Driven piles typically used by SCDOT include prestressed concrete, steel H-piles, steel pipe piles and combination piles consisting of concrete and steel H-pile sections. In addition, SCDOT has used timber piles in the past; however, timber piles are no longer allowed for the support of bridge structures. Concrete cylinder piles are currently being evaluated for inclusion in the pile types used by SCDOT. The use of concrete cylinder piles shall be approved in writing by SCDOT prior to commencing design. Piling is further categorized as either displacement or non-displacement. Displacement piles increase lateral ground stresses, densify cohesionless soils, can weaken cohesive soils (temporarily), have large set up times for fine-grained soils, and primarily get their capacity from skin friction. Typically prestressed concrete and closed-ended steel pipe piles are considered displacement piles. Non-displacement piles usually cause minimal disturbance to surrounding soil and primarily get their capacity from end bearing. Steel H-piles and opened steel pipe piles are considered non-displacement piles. According to the *Bridge Design Manual* (BDM) the minimum length of driven piling is 10 feet. The BDM provides typical sizes for driven piles. Table 16-2 provides a summary of these pile types and sizes.

Table 16-2, Typical Pile Types and Sizes

Pile Type	Size
Steel H-piles	HP 12x53 HP 14x73 HP14x89 HP 14x117[1]
Steel Pipe Piles	16-inch[2] 18-inch[2] 20-inch[2] 24-inch[2]
Prestressed Concrete Piles[3]	18-inch 20-inch 24-inch 30-inch[4] 36-inch[4]
Composite piles	18-inch with W 8x58 stinger 20-inch with HP 10x57 stinger 24-inch with HP 12x53 stinger

[1]used where penetration is minimal and nominal capacity is large
[2]wall thickness is ½ inch for all pipe pile sizes
[3]prestressed concrete piles are square in section
[4]these sizes are only allowed with the written approval of SCDOT

As required by AASHTO LRFD Bridge Design Specifications latest edition, driven pile analyses and design should address the following:

- Nominal axial resistance, pile type, size of pile group, and how the nominal axial pile resistance will be determined in the field;
- Pile group interaction;
- Pile penetration required to meet nominal axial resistance and other design requirements;
- Minimum pile penetration necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, liquefaction, lateral loads, and seismic conditions;
- Foundation deflection should meet the established movement and associated structure performance criteria;
- Pile foundation nominal structural resistance;
- Verification of pile driveability to confirm acceptable driving stresses and blow counts can be achieved, and;
- Long-term durability of the pile in service (i.e. corrosion and deterioration).

A thorough reference on pile foundations is presented in the FHWA publication *Design and Construction of Driven Pile Foundations – Volume I and II* (Hannigan et al, 2006).

16.3.1 Axial Compressive Capacity

There are numerous static analysis methods available for calculating the bearing capacity of a single pile. The axial compressive capacity for driven piles shall follow the procedures provide in the AASHTO LRFD Bridge Design Specifications (latest edition), Article 10.7 - Driven Piles. The methods found in the AASHTO LRFD Bridge Design Specifications are used to satisfy the Strength and Extreme Event limit states.

The basic LRFD equation presented previously and in Chapter 8 is expanded on the resistance side of the equation to account for the factored resistance of piles (R_r), and may be taken as:

$$\boldsymbol{Q} \le \varphi \boldsymbol{R_n} = \boldsymbol{R_r} = (\boldsymbol{R_t} + \boldsymbol{R_s})\varphi \qquad \text{Equation 16-2}$$

$$\boldsymbol{R_t} = \boldsymbol{q_t} * \boldsymbol{A_t} \qquad \text{Equation 16-3}$$

$$\boldsymbol{R_s} = \boldsymbol{q_s} * \boldsymbol{A_s} \qquad \text{Equation 16-4}$$

Where,

Q = Factored Load (demand)
R_r = Factored Resistance (i.e. allowable capacity)
R_n = Nominal Resistance (i.e. ultimate capacity)
R_t = Nominal Tip Resistance
q_t = unit tip resistance of pile (force/area)
A_t = area of pile tip (area)
R_S = Nominal Skin Friction Resistance
q_s = unit side resistance of pile (force/area)
A_s = surface area of pile side (area)
φ = Resistance Factor (see Chapter 9)

The nominal capacity of driven pile shall include the effects of scour. The nominal capacity shall be developed beneath the scour elevation or depth; however, the capacity developed in the scourable soils shall be determined and added to the nominal capacity to obtain the required ultimate bearing for use during pile installation.

The axial compressive design methodologies can be separated based on either total or effective stress methods or whether the soils are cohesionless or cohesive in nature. As indicated in the above equations the total axial compressive capacity of a deep foundation is based on the combination of unit side resistance and unit tip resistance values. Another factor that affects the axial compressive capacity of driven piles is the type of pile being installed (i.e. non-displacement vs. displacement). The followings methods shall be used to determine the capacity of driven piles:

(1) Nordlund Method: This method is an effective stress method and is used for sands and non-plastic silts (cohesionless soils). Further this method is based on field observations and considers the pile shape, and its soil displacement properties in calculating the shaft resistance. The unit shaft resistance is a function of: friction angle of the soil, the friction

angle of the sliding soils, pile taper, the effective unit weight of the soil, pile length, the minimum pile perimeter, and the volume of soil displaced. The friction angle of the soil shall be determined in accordance with the procedures outlined in Chapter 7. While there is no limiting value for the shaft resistance, the effective overburden pressure shall be limited to 3 kips per square foot (ksf). For pile sizes greater than 24 inches, this method tends to overpredict the pile capacity.

(2) α-Method: A total stress analysis used where the ultimate capacity is calculated from the undrained shear strength of the soil and is applicable for cohesive soils (i.e. clays and plastic silts). The undrained shear strength shall be determined in accordance with the procedures provided in Chapter 7. This method assumes that side resistance is independent of the effective overburden pressure and that the unit shaft resistance can be expressed in terms of an empirical adhesion factor times the undrained shear strength. The coefficient α depends on the nature and strength of the clay, pile dimension, method of pile installation, and time effects. The unit tip resistance is expressed as a dimensionless bearing capacity factor times the undrained shear strength. The dimensionless bearing capacity factor (N_c) depends on the pile diameter and the depth of embedment, and is usually assumed to be 9.

(3) SPT 97 Method: A total stress method originally developed by the Florida Department of Transportation (FDOT). The method uses uncorrected N_{60}-values to determine the ultimate capacity of driven piles. The method is based on the results of numerous load tests conducted by FDOT. The soils of the South Carolina Coastal Plain are similar to the soils in Florida. This is applicable to both cohesionless and cohesive soils.

(4) Historical Load Test Data: The ultimate capacity for driven piles may be developed based on the results of historical load test data from the anticipated load bearing stratum. The use of this type of data for development of capacity shall be reviewed by the GDSs and the PCS/GDS. The results of more than five load tests shall be used to develop the capacity. Load testing shall include static load tests, dynamic load tests and Statnamic load tests. A comparison to the soils at the load test site to the soils at the new location shall be performed.

For driven piles that will develop capacity in a layered subsurface profile consisting of both cohesionless and cohesive soils, the appropriate method will be used for each soil type and the nominal capacity determined by adding the results of the various layers together. For soil layers that are comprised of ϕ – c soils, the axial capacity for the layer should be determined using the Nordlund, SPT 97 and α methods with the actual capacity of the layer being the more conservative capacity.

The AASHTO LRFD Bridge Design Specifications provides additional methods for determining the axial compressive capacity of driven piles. These shall be used only as a check to the Norlund, SPT 97 and α methods discussed previously. These additional methods include:

(1) β-Method: An effective stress analysis used in cohesionless, cohesive, and layered soils.

(2) λ-Method: An effective stress method that relates undrained shear strength and effective overburden to the shaft resistance.

(3) Meyerhof SPT data Method: This method was derived by empirical correlations between Standard Penetration Test (SPT) results and static pile load tests for cohesionless soils.

(4) Nottingham and Schmertmann CPT Methods: The method uses Cone Penetrometer Test data relating pile shaft resistance to CPT sleeve friction.

In addition, Hannigan et al (2006) provides additional procedures for determining the axial compressive capacity of driven piles.

(1) Brown Method: An empirical method using SPT data for cohesionless materials.

(2) Elsami and Fellenius Method: A CPT based method that correlates the effective tip resistance to the unit shaft resistance.

(3) Laboratoire des Ponts et Chaussess (LPC) Method: A CPT based method that correlates the tip resistance, soil type, pile type and installation method to the unit shaft resistance.

As with the other methods listed in the AASHTO LRFD Bridge Design Specifications, these methods shall only be used to check the capacities determined by the Norlund, SPT 97 and α methods.

For driven piles bearing in rock with a RQD greater than 10 percent (see Chapter 6), the nominal capacity of the pile is typically limited by the structural capacity of the foundation element itself. This is especially true with prestressed concrete piles driven into rock, and why prestressed concrete piles typically have pile points when driven to bearing in rock. In many cases steel piles are fitted with "reinforced tips" to avoid damage to the foundation element.

There are numerous computer software packages available for performing the axial compressive capacity of driven pile foundations. The preferred software packages are DRIVEN as provided by the FHWA (www.fhwa.dot.gov/engineering/geotech/software/softwaredetail.cfm#driven) or SPT 97 as developed by the University of Florida for FDOT. The latest version of SPT 97 is contained within FB-Deep as developed by the University of Florida, Bridge Software Institute (http://bsi-web.ce.ufl.edu/products/). DRIVEN uses the Norlund and α methods for determining axial capacity for cohesionless and cohesive soils, respectively. FB-Deep can be applied to both cohesionless and cohesive soils. Other computer software packages may be used to determine axial compressive capacity of driven piles; however, prior to being used, the designer must submit copies of the output, the method used for design, a set of hand calculations performed using the procedure and evidence of applicability and acceptability using load testing information. This information shall be submitted to the PCS/GDS for technical review prior to being approved. It is incumbent upon the engineer, that prior to using any software, that the methodologies used by the software are fully understood.

16.3.2 Axial Uplift Capacity

The axial uplift capacity should be evaluated when tensile forces may be present. The side resistance of the driven pile shall be determined using either the Norlund or α methods. All capacity losses due to scour shall not be included in the determination of the axial uplift capacity. In addition, static settlement induced downdrag shall also not be included, since it is anticipated that at some point in time settlement will cease. The factored uplift resistance (R_r) may be evaluated by:

$$\boldsymbol{Q} \leq \varphi \boldsymbol{R_n} = \boldsymbol{R_r} = \varphi_{up} \boldsymbol{R_s}$$ Equation 16-5

Where,

Q = Factored Load (demand)
R_r = Factored Resistance (i.e. allowable capacity)
R_n = Nominal Resistance (i.e. ultimate capacity)
R_S = Nominal Skin Friction Resistance
φ and φ_{up} = Uplift Resistance Factors (see Chapter 9)

16.3.3 Group Effects

The analysis procedures discussed in the preceding paragraphs are for single driven piles. For most structures, driven piles are installed in groups. Typically SCDOT uses trestle bents (i.e. a single row of piles); these types of bents shall be considered to be groups for the purpose of determining group efficiency. The nominal axial (compressive or tensile) resistance of a pile group is the lesser of:

- The sum of individual nominal pile resistances, or
- The nominal resistance of the pile group considered as a block.

The minimum center-to-center spacing in a trestle bent is 2-1/2 times the nominal pile size; therefore, the group efficiency shall be taken as 1.0. For pile groups having more than two or more rows of piles, the group efficiency shall be determined following the procedures outlined in Article 10.7 of the AASHTO LRFD Bridge Design Specifications. The spacing between piles shall not be less than a center-to-center spacing of 2-1/2 times the nominal pile size in either the longitudinal or transverse directions. The procedures for determining the dimensions of the block are presented in the following section.

16.3.4 Settlement

Typically, the settlement of deep foundations is comprised of immediate and primary consolidation settlement and elastic compression (shortening). Secondary compression is not normally considered as part of the settlement of deep foundation. In many cases primary consolidation settlement is not a concern, since most deep foundations are founded in cohesionless soils, overconsolidated (OCR $\geq$ 4) soils, or rock. Elastic compression is included since the deep foundation will elastically deform when a load is applied. Pile groups are used in determining the amount of settlement instead of single piles, since very rarely are single piles used to support a structure. The total settlement is defined by the following equation.

$$\Delta_V = S_t = S_i + S_c + S_s + \Delta_E \qquad \text{Equation 16-6}$$

Where,

S_t = Δ_V = Total Settlement
S_i = Immediate Settlement
S_c = Primary Consolidation Settlement
S_s = Secondary Compression Settlement
Δ_E = Elastic Compression

Elastic compression is the compression (deflection or shortening) of a single pile caused by the application of load at the top of the pile. The elastic compression of combination piles is complex or difficult to determine. Therefore, engineering judgment should be used in determining if the concrete or steel portion of the composite pile contributes more to the settlement of the pile group or not. Elastic compression should be determined using the following equation.

$$\Delta_E = \frac{Q_a L}{AE} \qquad \text{Equation 16-7}$$

Where,

Q_a = Applied load
L = Pile length (embedment)
A = Cross sectional area of pile
E = Elastic modulus of pile material

For piles founded in cohesionless soils and in overconsolidated (OCR $\geq$ 4) cohesive soils, the settlement shall be determined using elastic theory as presented in Chapter 17. An equivalent foundation is used to determine the dimensions required. The width of the foundation (B_f) is either the pile diameter or face dimension for pile bents or the center to center of the outside piles along the shortest side of a pile footing (group). The length (L_f) is measured from the center to center of the outside piles along the length of the pile bent or pile footing. The depth of the equivalent foundation shall be two-thirds of the pile embedment depth. The applied bearing pressure (q_o) shall be taken as the sum of the pile service loads divided by the area of the equivalent footing. For each subsequent layer, the equivalent foundation is enlarged one horizontal to two vertical (1H:2V) portion until the settlement for all subsequent layers is determined.

The settlements for pile foundations placed in NC to slightly OC (1 < OCR <4) plastic cohesive soils shall be determined using consolidation theory as presented in Chapter 17. Similar to the elastic settlement determination an equivalent foundation shall be placed two-thirds of the pile embedment depth and the applied bearing pressures and changes in stress determined according. The applied bearing pressure (q_o) shall be taken as the sum of the pile service loads divided by the area of the equivalent footing. For each subsequent layer, the equivalent foundation is enlarged one horizontal to two vertical (1H:2V) portion until the settlement for all subsequent layers is determined.

16.3.5 Pile Drivability

Pile drivability refers to the ability of a pile to be driven to a desired penetration depth and/or capacity. Pile drivability must be performed as part of the design process. When evaluating drivability, the soil disturbance during installation and the time dependent soil strength changes should be considered.

There are three methods available for predicting and/or checking pile drivability.

- Wave Equation Analysis
- Dynamic Testing and Analysis
- Static Load Tests

Geotechnical Resistance factors for each of these three methods for analysis and level of capacity determination are provided in Chapter 9.

Wave equation analysis is required during design and again during construction. The following graphic illustrates some of the variables involved with the model. The most widely accepted program is GRLWEAP, and is available at http://pile.com/pdi/. It is incumbent upon the engineer, that prior to using any software, that the methodologies used by the software are fully understood.

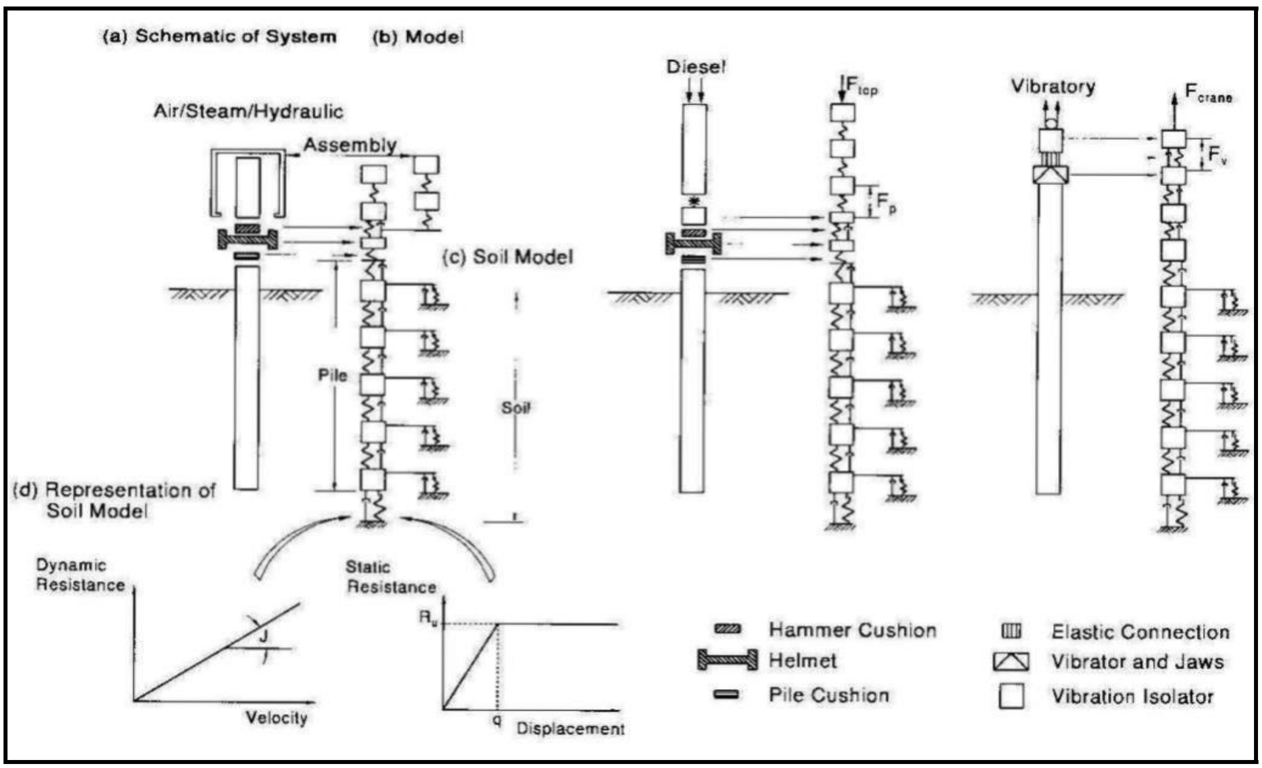


Figure 16-2, Typical Wave Equation Model (Hannigan, et al. 2006)

Some of the parameters that must be considered are hammer types, cushion material, pile properties and sizes, soil resistance distributions, soil quake and damping parameters. Some of these parameters are placed on the drawings (see Table 16-3). The wave equation is a computer simulation of the pile driving process that models wave propagation through the hammer-pile-soil system. The Ultimate Driving Resistance (see Foundation Length below) shall be used in wave equation analyses.

Table 16-3, Drivability Analysis

Skin Quake (QS)	0.10 in
Toe Quake (QT)	0.08 in
Skin Damping (SD)	0.20 s/ft
Toe Damping (TD)	0.15 s/ft
% Skin Friction	80%
Distribution Shape No.[1]	1
Bearing Graph	Proportional[2]
Toe No. 2 Quake	0.15 in
Toe No. 2 Damping	0.15 s/ft
End Bearing Fraction (Toe No. 2)	0.95
Pile Penetration	80%
Hammer Energy Range	25 – 60 ft-kips

[1]Distribution Shape No. varies with depth: 0 at the ground surface (creek bottom); 1 at a depth of 5ft; and 1 to a depth beyond driving depth below the ground surface.
[2]Bearing Graph options – proportional, constant skin friction, constant end bearing
Note: GRLWEAP (XXXX) was used to perform the wave equation analysis.

During construction, additional wave equation analysis should be performed on the actual driving system and cushions to be used. The model should be checked for adequate hammer energy, establish fuel settings, check compressive and tensile stresses, and to see if the blow counts fall within a certain range. The required number of blows should range from 36 to 180 blows per foot for the driving system to be acceptable. Practical refusal is defined as 5 blows per quarter (1/4) inch or 20 blows per inch.

Dynamic Testing and Analysis should be in accordance with ASTM D4945. This test consists of measuring strain and acceleration near the pile top during driving, or restrike using a Pile Driving Analyzer (PDA). The PDA is used to calculate valuable information such as pile driving stresses, energy transfer, damping and quake values, and the ultimate pile capacity. Additional analysis of the data collected in the field can be performed by using signal matching methods such as CAPWAP. Additional information on the dynamic testing is provided in Chapter 24.

Static load tests are the most accurate method of determining the ultimate resistance of a pile (if carried to failure). While this method accurately determines the obtained bearing and the required penetration resistance to achieve bearing, it does not determine if there is any damage to the pile during installation. If static load testing is recommended for a project with driven piles, then dynamic testing and Wave Equation analysis will also be required. This procedure has limited applicability since static load testing requires several days to setup and perform the testing. Static load testing can add several weeks to a construction project. Optimally, static load testing should be performed as a part of the design phase of a project, when the results can more readily be used to affect the design. A comprehensive report by the FHWA on this topic is *Static Testing of Deep Foundations* (Kyfor, et al., 1992).

16.4 DRILLED SHAFTS

A drilled shaft (also called drilled caisson or caisson) is a deep foundation element that is constructed by excavating a hole with power auger equipment. Reinforcing steel and concrete are then placed within the excavation. In unstable soils, casing or drilling slurry is used to maintain the stability of the hole. Drilling slurry typically consists of natural materials (i.e. bentonite); the use of polymer materials is not allowed. For certain geologic conditions (i.e. sound rock) the use of plain water (potable) as a drilling fluid is allowed; however, permission to use plain water must be obtained from SCDOT. Drilled shafts should be considered when large loads are anticipated (compressive, uplift or lateral) and where the amount of allowable deformation is small. Additionally, drilled shafts should be considered in locations where the losses due to scour are large, seismically induced downdrag loads are large or where the instability of slope cannot be maintained using conventional methods. Further drilled shafts should be considered when there is a limitation on water crossing work.

Drilled shaft sizes (diameters) can typically range from 30 inches (2-1/2 feet) to 144 inches (12 feet). Drilled shaft diameters are normally 6 inches larger than the column above the shaft or 6 inches larger than the rock socket below the shaft. Drilled shaft sizes typically used by SCDOT range from 42 inches (3-1/2 feet) to 84 inches (7 feet) in diameter. According to the BDM drilled shafts are typically used when the span length of a bridge is greater than 50 feet.

As required by AASHTO, the drilled shaft analyses and design should address the following:

- Nominal axial resistance of a single shaft and of a group of shafts.
- The resistance of the underlying strata to support the load of the shaft group.
- The effects of constructing the shaft(s) on adjacent structures.
- Minimum shaft penetration necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, liquefaction, lateral loads, and seismic conditions.
- Drilled Shaft nominal structural resistance
- Satisfactory behavior under service loads
- Long-term durability of the shaft in service (i.e. corrosion and deterioration)

A thorough reference on shaft foundations is presented in the FHWA publication *Drilled Shafts: Construction Procedures and Design Methods* (O'Neil and Reese, 1999).

16.4.1 Axial Compressive Capacity

There are numerous static analysis methods available for calculating the bearing capacity of a single drilled shaft. The axial compressive capacity for drilled shafts shall follow the procedures provide in the AASHTO LRFD Bridge Design Specifications (latest edition), Article 10.8 – Drilled Shafts. The methods found in the AASHTO LRFD Bridge Design Specifications are used to satisfy the Strength and Extreme Event limit states.

The basic LRFD equation presented previously and in Chapter 8 is expanded on the resistance side of the equation to account for the factored resistance of piles (R_r), and may be taken as:

$$\boldsymbol{Q} \leq \varphi\, \boldsymbol{R_n} = \boldsymbol{R_r} = \varphi_t \boldsymbol{R_t} + \varphi_s \boldsymbol{R_s} \qquad \text{Equation 16-8}$$

$$\boldsymbol{R_t} = \boldsymbol{q_t} * \boldsymbol{A_t} \qquad \text{Equation 16-9}$$

$$\boldsymbol{R_s} = \boldsymbol{q_s} * \boldsymbol{A_s} \qquad \text{Equation 16-10}$$

Where,

Q = Factored Load (demand)
R_r = Factored Resistance (i.e. allowable capacity)
R_n = Nominal Resistance (i.e. ultimate capacity)
R_t = Nominal Tip Resistance
q_t = unit tip resistance of pile (force/area)
A_t = area of pile tip (area)
R_S = Nominal Skin Friction Resistance
q_s = unit side resistance of pile (force/area)
A_s = surface area of pile side (area)
φ, φ_t and φ_s = Resistance Factors (see Chapter 9)

Where construction (permanent) casing is used to maintain the bore hole, the skin friction along the length of the casing shall not be included in the nominal or factored loads. Construction casing should normally be used on all drilled shafts in order to facilitate column construction above the shaft. The nominal resistance should be increased to account for the weight of the column above the top of the drilled shaft, if the provided loads are applied at the bent cap. If however the loads are applied at the top of the drilled shaft then the column weight should not be included.

The axial compressive design methodologies can be separated based on either total or effective stress methods or whether the soils are cohesionless or cohesive in nature. As indicated in the above equations the total axial compressive capacity of a deep foundation is based on the combination of unit side resistance and unit tip resistance values. The factored tip resistance shall be reduced to limit the amount of settlement of the drilled shaft; therefore, satisfying the Service limit state for the drilled shaft. See Chapter 17 for settlement analysis methods.

Provided below are the methods to be used to determine unit side resistances in soils:

(1) α-Method: A total stress analysis used where ultimate capacity is calculated from the undrained shear strength of the soil (clay or plastic silt). This approach assumes that side resistance is independent of the effective overburden pressure and that the unit shaft resistance can be expressed in terms of an empirical adhesion factor times the undrained shear strength. The coefficient α is related to the undrained shear strength and is derived from the results of full-scale pile and drilled shaft load tests. The top five feet and bottom one diameter should be ignored in estimating the nominal shaft side resistance. The unit tip resistance is expressed as a dimensionless bearing capacity factor times the undrained shear strength. The dimensionless bearing capacity factor (N_c) depends on the shaft diameter and the depth of embedment, and is usually assumed to be less than 9.

The AASHTO LRFD Bridge Design Specifications provides limitation on the shaft resistance determined using this method; these limitations shall be applied to all projects as required. The exception is if a construction casing is used, the shaft resistance shall be determined from the bottom of the casing to within 1 diameter of the bottom of the shaft.

(2) β-Method: An effective stress analysis used in cohesionless soils. The unit shaft resistance is expressed as the average effective overburden pressure along the shaft times the beta coefficient. This load transfer coefficient is based on average SPT N_{60} blow counts in the design zones under consideration. The maximum unit shaft resistance should not exceed 4 ksf unless supported by load test data. The unit tip resistance is based on the average SPT N_{60} blow counts being less than or equal to 50 blows per foot (bpf).

(3) Shafts in Rock: The side-wall shear of drilled shafts in rock are based upon the uniaxial compressive strength of rock, the modulus of elasticity of intact rock and the rock mass, and the jointing characteristics of the fractured rock mass parameters (size and thickness). The side-wall resistance is based on the assumption that the side-wall is smooth. If the side-wall is roughened, then the side-wall shear will be greater. This increased side-wall resistance shall be confirmed by load testing.

(4) Shafts in IGM: Intermediate geomaterial is material that is transitional between soil and rock in terms of strength and compressibility, such as saprolite and weathered rock. For detailed design, the procedures are presented in the FHWA publication *Drilled Shafts: Construction Procedures and Design Methods* (O'Neil and Reese, 1999).

(5) Historical Load Test Data: The ultimate capacity for drilled shafts may be developed based on the results of historical load test data from the anticipated load bearing stratum. The use of this type of data for development of capacity shall be reviewed by the GDSs and the PCS/GDS. The results of more than five load tests shall be used to develop the capacity. Load testing shall include static load tests, dynamic load tests and Statnamic load tests. A comparison to the soils at the load test site to the soils at the new location shall be performed.

Provided below are the methods to be used to determine unit tip resistances in soils:

(1) A total stress analysis method is used to determine the ultimate unit tip resistance capacity and is calculated from the undrained shear strength of a cohesive soil (clay or plastic silt). This method limits the unit tip resistance to 80 ksf and is based on the undrained shear strength of the soil located within 2 diameters of the tip of the shaft.

(2) The ultimate unit tip resistance of cohesionless soils is determined using a total stress analysis method. The method is based on the N_{60} and is limited to 60 ksf.

(3) The ultimate unit tip resistance for rock is based on the quality and strength of the rock within 2 diameters of the tip. The tip resistance shall be based on either the strength of the rock or the strength of the concrete whichever is lower.

The analysis procedures discussed in the preceding paragraphs are for single drilled shafts. For some structures, drilled shafts are sometimes installed in groups. Drilled shaft groups installed in cohesive and cohesionless soils will typically have group efficiencies less than 1 with spacings less than 6 and 4 diameters, respectively. The efficiencies of shaft groups are typically less than 1 due to overlapping zones of shear deformation and because the construction process tends to relax the effective stresses.

SCDOT recommends the resistance factor provided in Chapter 9 for analysis for drilled shaft group capacity in clays. This resistance factor is based on block failure of the clays, which is more due to settlement of the group. There is no group resistance factor for sands other than reduction required for group spacing. For additional information on the analysis of drilled shaft groups please refer to Article 10.8 in the AASHTO LRFD Bridge Design Specifications (latest edition) or the FHWA publication *Drilled Shafts: Construction Procedures and Design Methods* (O'Neil and Reese, 1999).

SHAFT v5.0 (Ensoft, Inc. at http://www.ensoftinc.com/) – This is a windows-based program used to compute the axial capacity and the short-term, load versus settlement curves of drilled shafts in various types of soils. ***SHAFT*** *v5.0* can analyze drilled-shaft response in six types of strata: i) clay - cohesive geomaterial, ii) sand - cohesionless geomaterial, iii) clay-shale, iv) strong rock, v) gravel - cohesionless IGM, and vi) weak rock - cohesive IGM. The program allows for any combination of soil layers to be placed in a layered profile. Most of the analytical methods used by ***SHAFT*** are based on suggestions from the latest FHWA manual *Drilled Shafts: Construction Procedures and Design Methods* (O'Neil and Reese, 1999). It is incumbent upon the engineer, that prior to using any software, that the methodologies used by the software are fully understood.

16.4.2 Uplift Capacity

The uplift capacity should be evaluated when there are chances that upward forces may be present. The pile side resistance should be determined from one of the methods presented above. The factored uplift resistance (R_r) may be evaluated by:

$$\boldsymbol{Q} \leq \varphi \boldsymbol{R_n} = \boldsymbol{R_r} = \varphi_{up} \boldsymbol{R_s} \qquad \text{Equation 16-11}$$

Where,

Q = Factored Load (demand)
R_r = Factored Resistance (i.e. allowable capacity)
R_n = Nominal Resistance (i.e. ultimate capacity)
R_S = Nominal Skin Friction Resistance
q_s = unit side resistance of pile (force/area)
A_s = surface area of pile side (area)
φ and φ_{up} = Resistance Factors (see Chapter 9)

Shaft group uplift resistance is the lesser of:

- The sum of the individual shaft uplift resistance, or
- The uplift resistance of the shaft group considered as a block.

16.4.3 Group Effects

The analysis procedures discussed in the preceding paragraphs are for single drilled shafts. For most structures, drilled shafts are installed in groups. Typically SCDOT uses frame bents (i.e. a single row of drilled shafts with a column on top of each shaft); these types of bents shall be considered to be groups for the purpose of determining group efficiency. The effect of excavating and concreting drilled shafts adjacent to existing drilled shafts can cause a reduction in effective stresses; therefore causing a reduction in the capacity of the existing drilled shaft. The nominal axial (compressive or tensile) resistance of a pile group is the lesser of:

- The sum of individual nominal drilled shafts resistances, or
- The nominal resistance of the drilled shaft group considered as a block.

The soil conditions that the drilled shafts are founded in affects the capacity of the group. The minimum center-to-center spacing is 2-1/2 times the nominal drilled shaft size. A group efficiency factor (η) shall be taken as 0.65 and increases linear to 1.0 for a center-to-center spacing of 4 times the nominal shaft size. (see Equation 16-12)

$$\eta = \mathbf{0.2333}\boldsymbol{x} + \mathbf{0.0667} \qquad \text{Equation 16-12}$$

Where,

η = Group efficiency factor
x = Number of diameters in center-to-center spacing (i.e. x=2.5)

For center-to-center spacings greater than 4 times the nominal drilled shaft size use η = 1.0.

For pile groups having more than two or more rows of drilled shafts, the group efficiency shall be determined following the procedures outlined in Article 10.8 of the AASHTO LRFD Bridge Design Specifications. The spacing between drilled shafts shall not be less than a center-to-center spacing of 2-1/2 times the nominal drilled shaft size in either the longitudinal or transverse directions. The procedures for determining the dimensions of the block are presented in the following section.

16.4.4 Settlement

Settlements of single drilled shafts under axial compression loadings (Service limit state) should be determined. Settlements determine the distribution of load caring capacity between side and tip resistances. Determine the distribution of load between the side and tip using *Appendix C: Estimation of Axial Movement of Drilled Shafts* in Drilled Shafts: Construction Procedures and Design Methods, FHWA-IF-99-025.

Typically, the settlement of deep foundations is comprised of immediate and primary consolidation settlement and elastic compression (shortening). Secondary compression is not normally considered as part of the settlement of deep foundation. In many cases primary consolidation settlement is not a concern, since most deep foundations are founded in cohesionless soils, overconsolidated (OCR $\geq$ 4) soils, or rock. Elastic compression is included since the deep foundation will elastically deform when a load is applied. Drilled shaft groups are used in determining the amount of settlement instead of single drilled shafts. However, in some cases (i.e. hammer heads) single drilled shafts are used to support a structure. The total settlement is defined by the following equation.

$$\Delta_V = S_t = S_i + S_c + S_s + \Delta_E \qquad \text{Equation 16-13}$$

Where,

S_t = Δ_V = Total Settlement
S_i = Immediate Settlement
S_c = Primary Consolidation Settlement
S_s = Secondary Compression Settlement
Δ_E = Elastic Compression

Elastic compression is the compression (deflection or shortening) of a drilled shaft caused by the application of load at the top of the drilled shaft. The elastic compression of drilled shafts is complex or difficult to determine. Therefore, engineering judgment should be used in determining the elastic properties of a drilled shaft. Elastic compression should be determined using the following equation.

$$\Delta_E = k\frac{Q_a L}{AE} \qquad \text{Equation 16-14}$$

Where,

Q_a = Applied load
L = Drilled shaft length (embedment)
A = Cross sectional area of drilled shaft
E = Elastic modulus of drilled shaft material
k = Factor that accounts for load distribution along drilled shaft (see Table 16-4)

Table 16-4, k Factor

Loading Condition	k Factor
All End Bearing[1]	1.00
All Side Resistance	0.50
Combination of End and Side	0.67

[1]Drilled shafts founded in rock are included in this category

For drilled shafts founded in cohesionless soils and in overconsolidated (OCR $\geq$ 4) cohesive soils, the settlement shall be determined using elastic theory as presented in Chapter 17. An equivalent foundation is used to determine the dimensions required. The width of the foundation (B_f) is either the drilled shaft diameter or the center to center of the outside shafts along the shortest side of a shaft footing (group). The length (L_f) is measured from the center to center of the outside shafts along the length of the shaft frame or shaft footing. The depth of the equivalent foundation shall be two-thirds of the drilled shaft embedment depth. The applied bearing pressure (q_o) shall be taken as the sum of the drilled shaft service loads divided by the area of the equivalent footing. For each subsequent layer, the equivalent foundation is enlarged one horizontal to two vertical (1H:2V) portion until the settlement for all subsequent layers is determined.

The settlements for drilled shaft foundations placed in NC to slightly OC (1 < OCR <4) plastic cohesive soils shall be determined using consolidation theory as presented in Chapter 17. Similar to the elastic settlement determination an equivalent foundation shall be placed two-thirds of the drilled shaft embedment depth and the applied bearing pressures and changes in stress determined according. The applied bearing pressure (q_o) shall be taken as the sum of the drilled shaft service loads divided by the area of the equivalent footing. For each subsequent layer, the equivalent foundation is enlarged one horizontal to two vertical (1H:2V) portion until the settlement for all subsequent layers is determined.

Once the total settlement (S_t or Δ_V) is determined, then the distribution of the load between side and end should be determined as indicated previously.

16.5 DRILLED PILES

Drilled piles are constructed normally at end bents where the depth to rock is less than ten to fifteen feet. Drilled piles can be a subset of drilled shafts or driven piles depending on the strength of the rock. An RQD of less than ten percent indicates that the pile may be driven; however, refusal criteria still apply (i.e. 5 blows in ¼ inch). The capacity of the drilled pile is determined based on whether the pile is driven or not after being placed in the bore hole. Piles placed in the bore hole and not driven should be designed using drilled shaft design procedures. This design methodology requires coordination with the structural designer to ensure that adequate load transfer from the steel to the concrete occurs. Drilled piles typically consist of steel H-piles having sizes of HP12x53 and HP14x73. The borehole shall be of sufficient diameter to allow for the insertion of the pile and the placement of concrete.

16.6 CONTINUOUS FLIGHT AUGER PILES

Continuous flight auger piles (CFAs) also known as Auger Cast Piles are a new technology being considered by FHWA for transportation projects. CFAs may be used on SCDOT projects; however, CFAs may not be used to support bridges. The use of CFAs on any SCDOT project must be approved prior to completion of preliminary design. Approval shall be in writing from either the Regional Production Engineer or the Preconstruction Support Engineer. In addition, the designer shall contact the PCS/GDS for instructions on analytical methods for determining capacity. CFAs will range in size from 18 to 30 inches (1-1/2 to 2-1/2 feet, respectively) in diameter for SCDOT projects.

16.7 MICROPILES

The latest version of the AASHTO LRFD Bridge Design Specifications allows for the use of micropiles to support structures. Article 10.9 of the AASHTO LRFD Bridge Design Specifications provides a list of when micropiles would be acceptable; however, approval by either the Regional Production Engineer or the Preconstruction Support Engineer shall be obtained prior to designing micropiles. The design of micropiles when allowed shall follow Article 10.9.

16.8 LATERAL CAPACITY

Designing for lateral capacities of deep foundations consists of either lateral load tests or analytical methods. Full scale load tests will not be discussed but analytical methods will be presented as an overview. More detailed information and the theory can be found in the FHWA publication *Handbook on Design of Piles and Drilled Shafts Under Lateral Load* (Reese, 1984).

Pile or shaft foundations must be designed to resist horizontal loads due to wind, traffic loads, bridge curvature, vessel or traffic impact, and seismic events. The movements or deflections as a result of the loadings should be within Performance Limits from Static or Seismic Loadings.

Methods of analysis that use manual computation include Broms' Method. Reese developed analysis methods that model the horizontal soil resistance using P-y curves.

Deep foundation horizontal movement at the foundation design stage may be analyzed using computer applications that consider soil-structure interactions. Computer programs are available for analyzing single piles and pile groups.

According to Hannigan, et al. (1998), the design of laterally loaded piles requires the combined skills of the geotechnical and structural engineer. It is inappropriate for the geotechnical engineer to analyze a laterally loaded pile without a full understanding of pile-structure interaction. Likewise it is inappropriate for the structural engineer to complete a laterally loaded pile design without a full understanding of how pile section or spacing changes may alter the soil response. Because of the interaction of pile structural and geotechnical considerations, the economical solution of lateral pile loading problems requires communication between the structural and geotechnical engineer.

Lateral designs using static loadings are governed by the Service Limit State. For group loadings using the P-y method of analysis, P-multipliers should be used in accordance with AASHTO LRFD Bridge Design Specifications Article 10.7 – Driven Piles.

SCDOT has established a process once the final subsurface investigation has taken place and prior to issuance of the BGER. The BGER is used to design foundations for bridges and bridge related structures. For drilled shaft/pile bents and drilled shaft/pile group footings, the BGER provides estimated pile/shaft tip elevations, the minimum pile/shaft tip elevations required to maintain lateral stability (critical depth), and the necessary soil parameters to develop a P-y soil model of the subsurface that is used in performing foundation lateral soil-structure interaction analyses. The Design Team then performs the lateral soil-structure interaction analysis with computer programs such as LPILE or FB-Pier. The Design Team uses this information to compute lateral displacements and to analyze the structural adequacy of the columns and foundations. The lateral soil-structure interaction analysis is also used to select the appropriate method (point-of-fixity, stiffness matrix, linear stiffness springs, or P-y nonlinear springs) to model the bridge foundation in the structural design software.

If lateral design controls the minimum point of penetration for a deep foundation, the BGER should indicate this fact. In addition, for driven piles, the nominal capacity should be increased to account for the additional installation depth required to achieve the tip elevation governed by lateral design.

16.9 DOWNDRAG

Downdrag loads (also known as Negative Skin Resistance) can be imposed on piles and shafts where:

- Sites are underlain by compressible material such as clays, silts, or organic soils,

- Fill will be or has recently been placed adjacent to the piles or shafts, such as is frequently the case for bridge approach fills,
- The groundwater is substantially lowered, or,
- Liquefaction of loose sandy soils can occur.

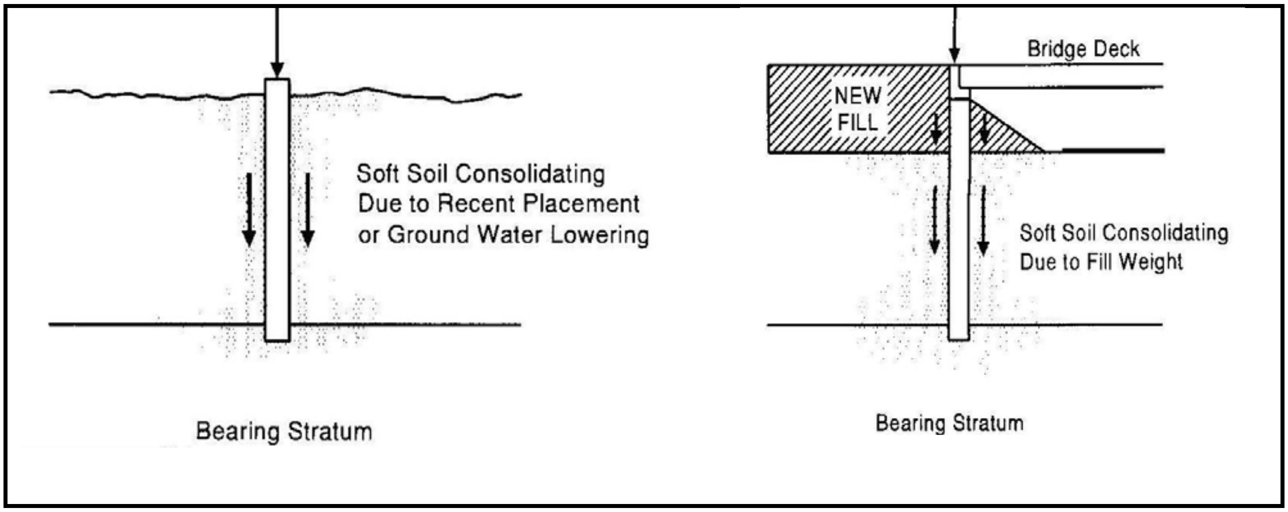


Figure 16-3: Downdrag Scenarios due to Compressible Soils and from Embankment Fills (Hannigan, et al. 1998)

The effect of downdrag loads can be mitigated thourgh the use of embankment surcharge loads, ground improvement techniques, and/or vertical drainage and settlement monitoring measurements. In addition, either coatings or sleeves/jackets may be applied to the piles allowing the soil to slide adjacent to the piles.

The following steps should be followed in determining the force effects as a result of downdrag:

(1) Establish a soil profile and properties for computing settlement.

(2) Perform settlement computation for the soil layers along the length of the pile or shaft using the procedures outlined in Chapter 17.

(3) Determine the length of pile or shaft that will be subject to downdrag. If the settlement in the soil layer is 0.4 inches or greater relative to the pile or shaft, downdrag can be assumed to fully develop; therefore, the use of residual shear strengths is required for all soils above the soil layer that experiences 0.4 inches of vertical movement.

(4) Determine the magnitude of the Downdrag (DD) load by computing the negative skin resistance (Q_{sdd}) using static analysis procedures for cohesionless or cohesive soils as described previously for piles and shafts. Sum the negative skin resistance for all layers contributing to downdrag from the lowest layer to the bottom of pile cap or ground surface.

(5) Apply load factors (γ_p) to the Downdrag (DD) using the load factors found in Chapter 8 for static analysis. For seismically induced Downdrag use a load factor of 1.05.

(6) For statically induced Downdrag, the Downdrag load should be added to the demand and the required factored resistance determined. This should only be done if no methods have been used to compensate for the Downdrag load (i.e. bitumen covering on pile or surcharging of the site prior to pile installation, therefore reducing relative settlement between the soil layer and foundation to less than 0.4 inches).

(7) For seismically induced Downdrag, the Downdrag load is added to the demand and compared to the nominal resistance (i.e. resistance factor (φ) is 1.0). If the nominal resistance is greater, the driven pile or drilled shaft is acceptable. If the nominal load is less, then the driven pile or drilled shaft must be extended to obtain additional resistance to equalize demand and nominal resistance.

(8) The resistance in the soil layers that will experience Downdrag should not be included in the development of the nominal resistance of the foundation system.

(9) For driven piles, the amount of Downdrag should be added to the nominal resistance to obtain a resistance required for pile installation (i.e. Ultimate Driving Resistance). This resistance is only used in WEAP analysis and pile installation. Be sure to include any time dependent effects from the soil layer experiencing downdrag.

16.10 FOUNDATION LENGTH

As part of the design process the geotechnical engineer shall determine the anticipated minimum tip elevation required to achieve the required capacity. The report shall clearly indicate the governing conditions for development of the tip elevation using the words depicted in Table 16-5.

Table 16-5, Governing Conditions

Loading Type	Loading Direction
Static	Axial (Compression or Tensile)
Seismic	Lateral

Each governing condition shall consist of a loading type and a loading direction (i.e. Seismic Lateral or Static Lateral). In addition, to indicating which governing condition was used to develop the minimum tip elevation, the report shall also include a loading table that will provide the information depicted in Table 16-6, Pile Bearing or Table 16-7, Drilled Shaft Bearing.

Table 16-6, Pile Bearing

Factored Design Load	56 tons
Geotechnical Resistance Factor	0.40
Nominal Resistance	140 tons
Estimated Scour	20 tons
Liquefaction-induced Downdrag	10 tons
Required Driving Resistance	170 tons

Table 16-7, Drilled Shaft Bearing

Factored Design Load	370 tons
Factored Resistance – Side	370 tons
Factored Resistance – End	0
Geotechnical Resistance Factor – Side	0.50
Geotechnical Resistance Factor – End	0.50
Total Nominal Resistance	740 tons

The Ultimate Driving Resistance is used to determine the driving resistance (see Pile Drivability above) and acceptability of the driving equipment. Depending on the controlling condition this driving resistance could consist of the Nominal Resistance plus the Estimated Scour (as indicated in Table 16-4) or the driving resistance could be the Resistance required to achieve a required minimum tip elevation (i.e. the Nominal Resistance is achieved prior to reaching minimum tip elevation). In these cases the piles will be driven to a higher capacity then required to achieve the Nominal Resistance and the Pile Drivability analysis shall account for this higher required resistance. In addition, this may effect the pile driving equipment that a contractor selects.

Please note that the weight of a drilled shaft is not subtracted from the nominal capacity, since the geotechnical resistance factors were obtained from static load tests. Therefore the resistance factors already account for the weight of the shaft in both compression and tension. However, depending on where the loads are applied, the weight of the column above the drilled shaft may need to be added to the axial load. The column weight is added if the loads are applied at the top of the column, however, if the loads are applied at the top of the shaft, the column weight is not added. The factored column weight shall be determined by the structural designer and provided to the geotechnical engineer. In addition, the structural designer shall indicate where the loads are applied on the load data sheet.

If the Downdrag loads exceed the Nominal Resistance of the deep foundation, then additional length will be required. For driven piles this additional length shall be accounted for in the Ultimate Driving Resistance. For drilled shafts that tip elevation shall be changed to reflect this increase and an Ultimate Bearing Resistance shall be indicated on the plans.

16.11 REFERENCES

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Chapter 17

EMBANKMENTS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

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CHAPTER 17

EMBANKMENTS

17.1 INTRODUCTION

This Chapter provides general guidance in stability and settlement design and analysis of earthen embankments (unreinforced slopes) and earth retaining structures (ERS) (see Chapter 18) belonging to SCDOT. This Chapter is concerned with the external stability of slopes and ERSs. The internal stability of ERSs is the responsibility of the structural designer. The settlement of earthen embankments, ERSs, foundations (shallow and deep) is discussed in this Chapter. These settlements are for the Service limit state. Settlements caused by the Extreme Event I limit state are discussed in Chapter 13. The amount of total and differential settlement and the rate of settlement shall be determined. All settlements will be determined for a twenty-year period, unless specifically directed by either the GDS or the PCS/GDS to use another time period.

Stability and settlement should be determined on the critical section. The selection of the critical section or sections is left to the geotechnical engineer-of-record. However, the following are suggested guidelines for use in the selection process:

1. Highest slope or ERS
2. Steepest slope
3. Soft underlying soils
4. Slope or ERS critical to performance of a structure (i.e. bridge, culvert, etc.)

Embankments with heights less than 5 feet and slopes 2H:1V or flatter do not require analysis, unless in the opinion of the geotechnical engineer-of-record the analysis is required.

Slopes and ERSs can be divided into two main categories; natural and man-made. Natural slopes are those slopes formed by natural processes and are composed of natural materials. Natural slopes can include river banks to the valleys passing through or parallel to mountain ridges. Man-made slopes are those slopes that are constructed by man. Man-made slopes may be subdivided into two types of slopes, fill (bottom up construction) and cut (top down construction). Fill slopes, including ERSs, are constructed by placing soil materials to elevate the grade above the natural or existing grade. Fill slopes may be unreinforced or reinforced (see Chapter 18). Cut slopes, including ERSs, are constructed by excavating material from either a natural or man-made fill slopes in order to reduce the grade. The stability and settlement procedures discussed in this Chapter exclusively apply to slopes constructed of soil and founded soil or rock materials. For the design of slopes in rock see FHWA NHI-132035 – *Rock Slopes* for design procedures.

17.2 FAILURE MECHANISMS

There are several failure mechanisms that affect slopes and ERSs. The mechanism of failure may dictate the required analysis method to be used to determine stability or instability. Further, the type of soil that the slope or ERS is comprised of will also affect the failure mechanism. The different failure mechanisms are listed below:

1. Creep
2. Flow
3. Fall and Topple
4. Slide
5. Spread
6. Deformation and settlement

17.2.1 Creep

Creep is the very slow movement of slopes, either natural or man-made, toward the toe and a more stable configuration. This movement can range up to approximately one inch per year. Slopes that creep can remain stable for extended periods of time. However, once the limit of the soil shear strength has been reached, the amount of movement may increase and the time for movement may decrease resulting in a rapid or sudden failure of the slope. Creep movements can be divided into two general types, seasonal and massive. Seasonal creep is the creep that occurs during successive seasons, such as freezing and thawing, or wetting and drying. The amount of seasonal creep can vary from year to year, but is always present. Seasonal creep extends to the depth limit of seasonal variations of moisture and temperature. Massive creep causes almost constant movement within the slope and is not affected by seasonal variations. Massive creep typically occurs in clay-rich soils. While the actual mechanism of massive creep is not fully understood, this type of creep can be attributed to exceeding some threshold shear strength that is below peak shear strength. This threshold shear strength may be a very small portion of the peak shear strength. If the stresses in the slope remain below the threshold level, then movement will not occur; however, if the stresses exceed the threshold, then movements will occur. If enough stresses accumulate to exceed the peak shear strength, then a more rapid failure is possible. In general, once creep has started it is difficult or impossible to stop. However, the rate of creep may be reduced by placement of drainage. During the geoscoping of the project site, the trees should be observed for any convex curvature with the convex part pointing down slope (see Figure 17-1).

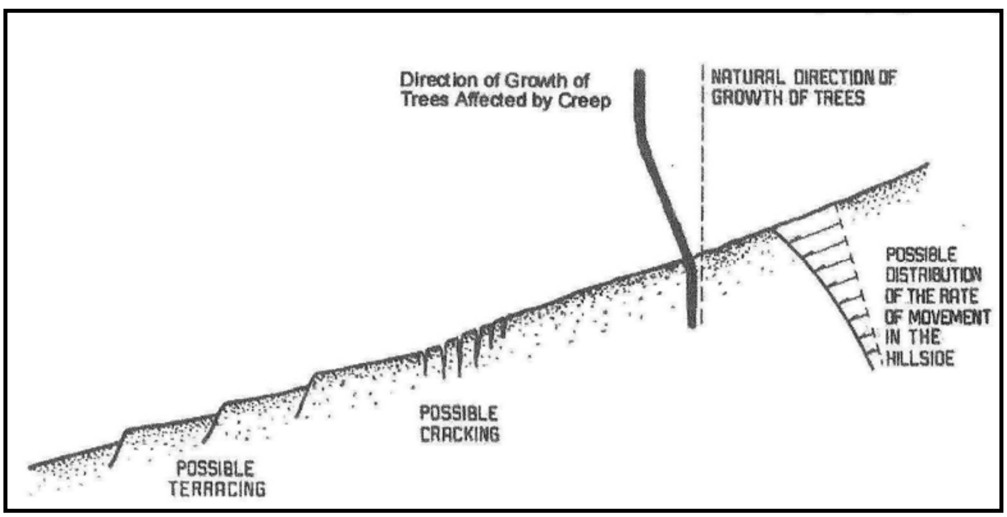


Figure 17-1, Signs of Creep
(Soil Slope and Embankment Design – September 2005)

17.2.2 Slide

Slides are downward slope movements that occur along definite slip or sliding surfaces. Slides may be translational, rotational, or a composite of rotation and translation. Translational slides are typically shallow and linear in nature. This type of slide typically occurs along thin weak layers or along the boundary between a firm overlying layer and weaker underlying layer (see Figure 17-2).

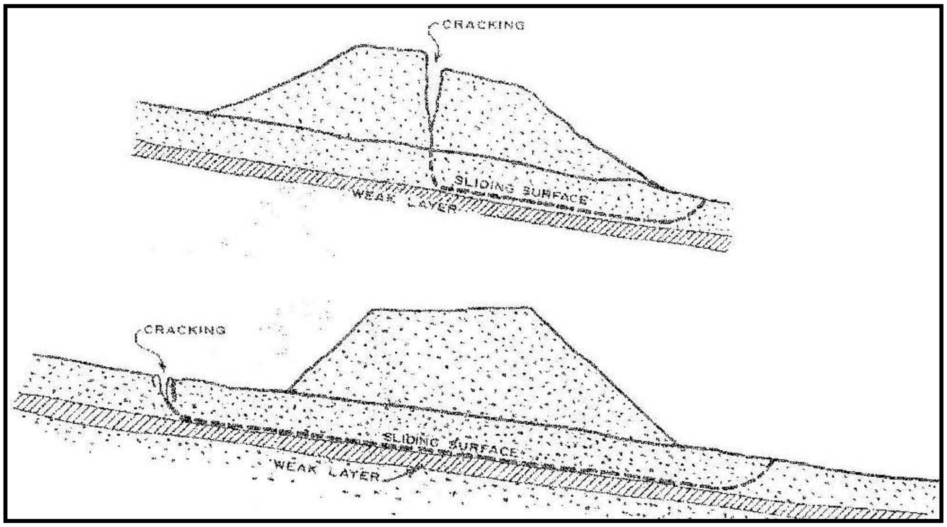


Figure 17-2, Translational Slide
(Soil Slope and Embankment Design – September 2005)

Rotational slides are slides that form an arc along the shearing surface. This is the most common type of failure analyzed. In soft, relatively homogenous cohesive materials, the rotational slide forms a deep seated arc, while in cohesionless materials; the rotational slide failure surface tends to be relatively shallow. Examples of different types of rotational slides are depicted in Figure 17-4.

Compound slides are a composite of translational and rotational slides. This type of slide tends to have a complex structure and can be difficult to analyze. Compound slides can have two general forms, retrogressive and progressive. Retrogressive compound slides continue to cut into the existing slope. After initial failure, the new slope that is formed is unstable and fails, developing another new unstable slope face that fails. This slide type may result in a series of slides that tend to converge on one extended slope. Progressive slides occur when an existing slope surface is loaded with either new fill or debris from a slope failure, resulting in failure of the slope toward the toe. Compound slide types are depicted in Figure 17-3.

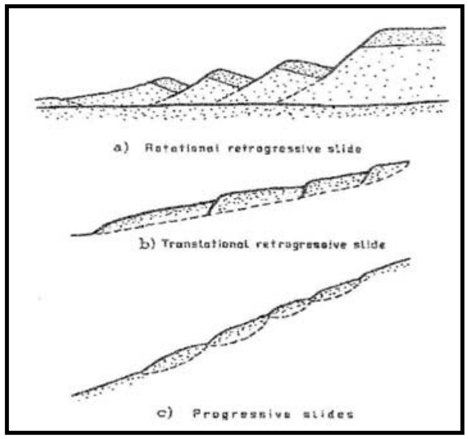


Figure 17-3, Compound Slide
(Soil Slope and Embankment Design – September 2005)

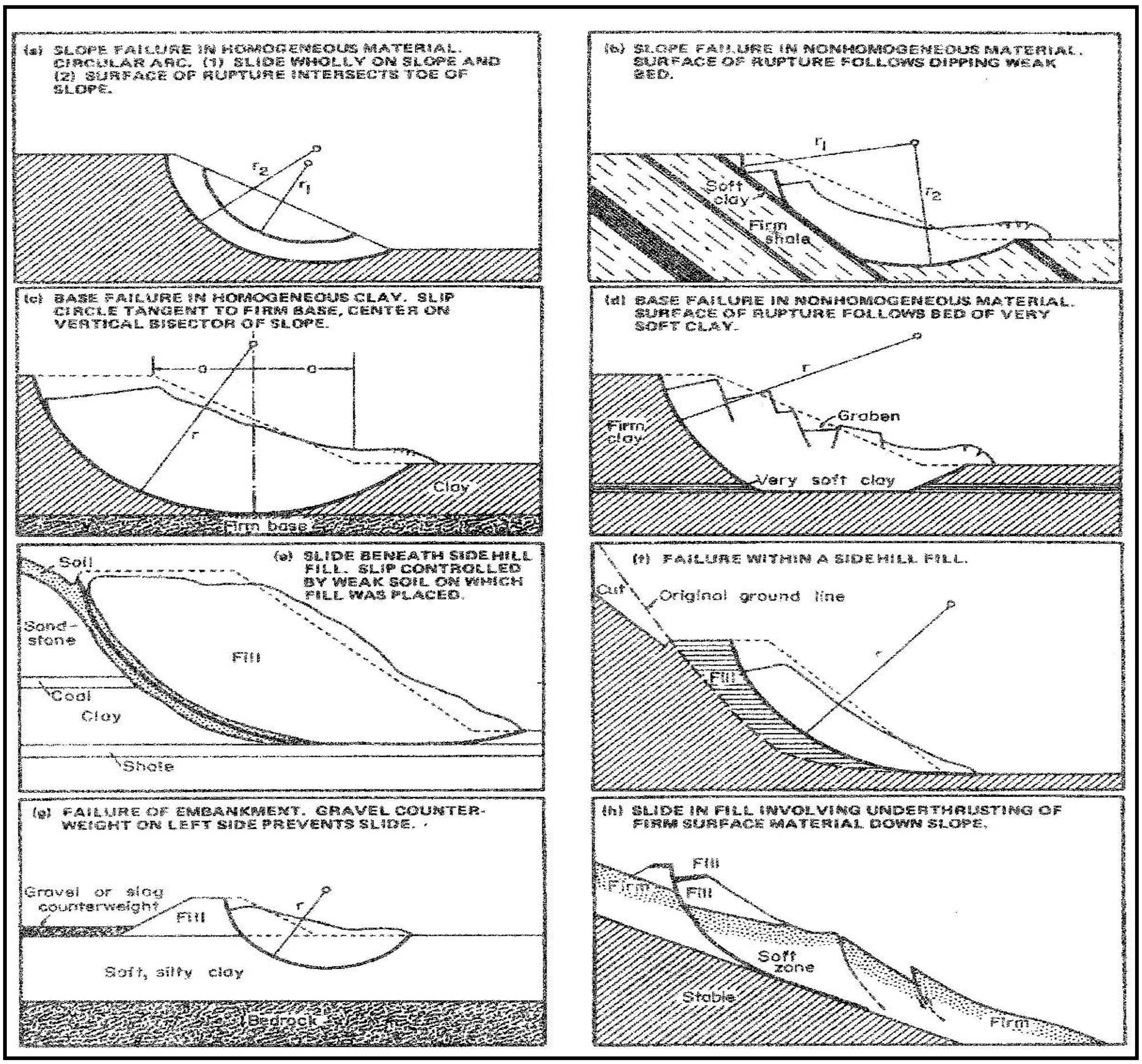


Figure 17-4, Rotational Slide
(Soil Slope and Embankment Design – September 2005)

17.2.3 Flow

Flow failures can occur in both dry, as well as wet soils, depending on the materials and the relative density. Flows are defined as mass soil movements that have greater internal deformations than slides. In a slide, the soil block will maintain some definition during sliding, whereas in flows, the definition of the block is completely lost. Flow failures, depending on the moisture condition of the soil, may behave similar to a fluid. In dry flow failures of fine-grained cohesionless soils, the movements are caused by a combination of sliding and individual particle movements. These types of failures may be caused by soils being cut on steep slopes that are stable when first constructed, but become unstable with time. Dry flow failures are also termed earthflows. Moist flows occur in soils that have higher moisture contents than the soils in a dry flow. In cohesive soils, moist flows occur when the moisture content exceeds the liquid limit of the material. In cohesionless soils, moist flows may occur when water becomes trapped in the soils by an impermeable barrier. Liquefaction is a form of moist flow that is caused by high moisture content and a seismic shock (see Chapter 13). Wet flows are also termed mudflows. See Figure 17-5 for dry and moist flow failure examples.

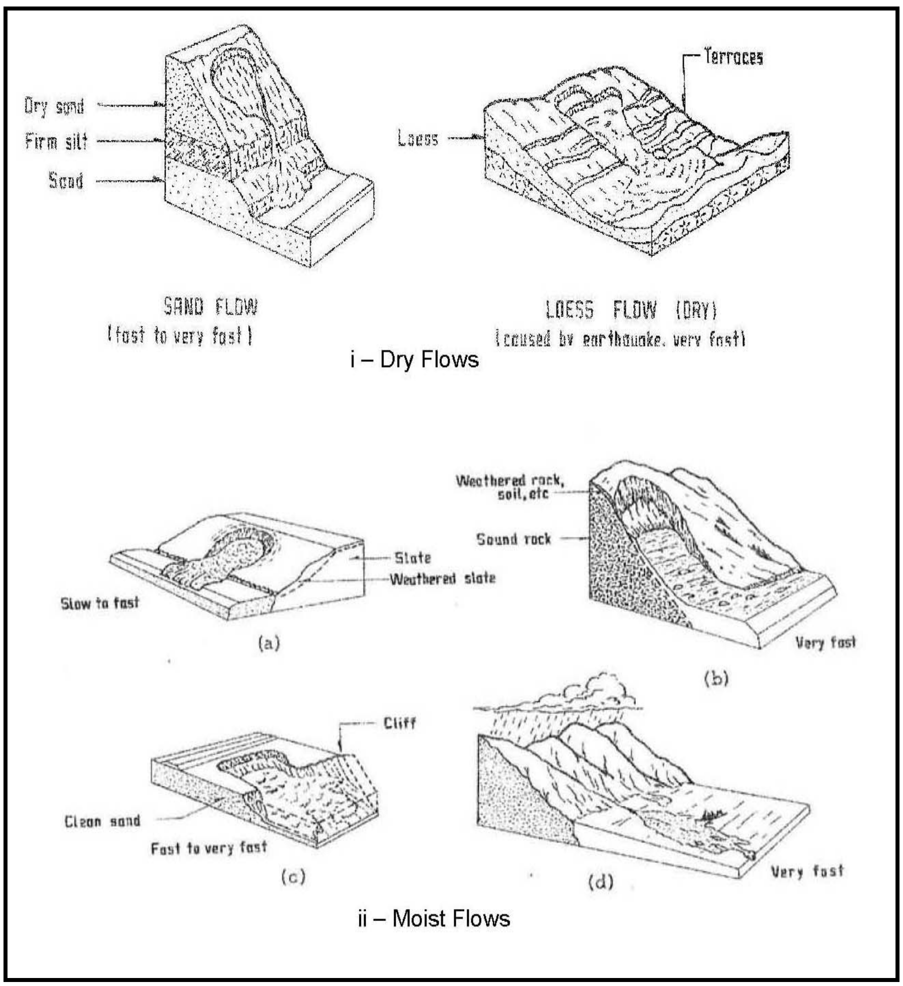


Figure 17-5, Flow Failures
(Soil Slope and Embankment Design – September 2005)

17.2.4 Spread

Spread was originally defined by Terzaghi and Peck in 1948 to describe sudden movements of water bearing seams of cohesionless materials overlain by homogeneous clays or fills. Spreads occur on very gentle slopes (< 5 percent) or flat terrain. Spreads can occur in cohesionless soils (liquefaction) or in cohesive soils (quick clays) that are externally loaded. In the case of liquefaction, the load is the seismic shock, and in quick clays, that load may be applied by the placement of fill materials. Figure 17-6 illustrates a typical soil spread.

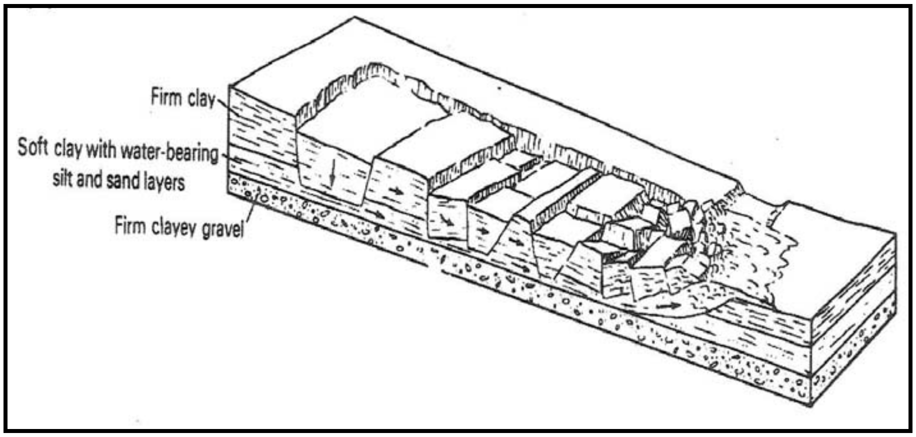


Figure 17-6, Spread Failure
(Soil Slope and Embankment Design – September 2005)

17.2.5 Fall and Topple

Fall and topple failures typically occur on rock slopes, although, topples can occur in steeply cut or constructed soil slopes. Falls are sudden movements of rocks and boulders that have become detached from steep slopes or cliffs (see Figure 17-7). Cracks can form at the top of the steep slope that may fill with water that will exert pressure on the rock mass causing it to fall. The water may freeze during colder weather exerting pressure on the rock mass as well. A topple is the forward rotation of rock or soil mass around a pivot point in the mass (see Figure 17-8). The steepness of the slope affects the formation of the topple, the slope can be constructed too steep or can be eroded to a steep configuration.

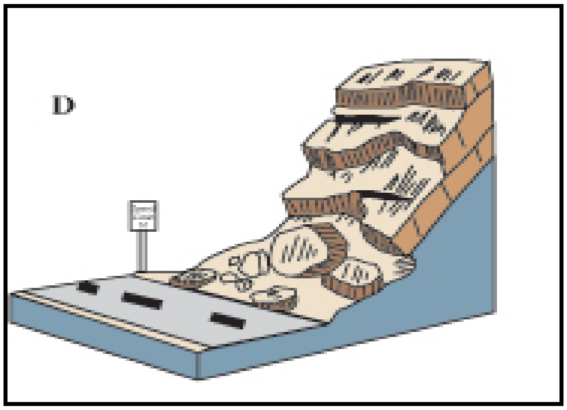


Figure 17-7, Fall Failure
(Landslide Types and Processes – July 2004)

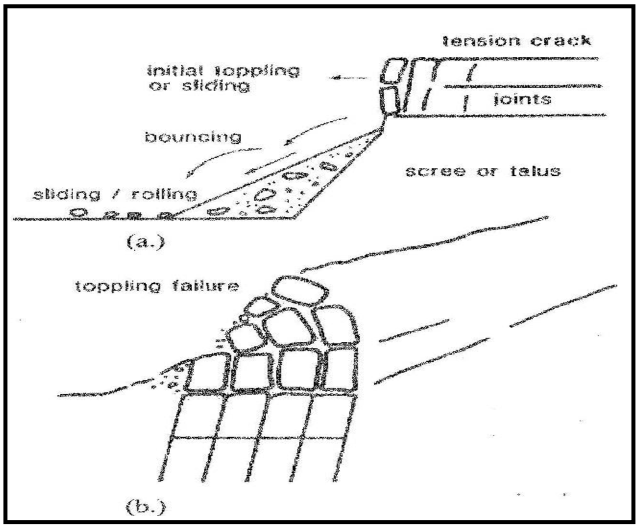


Figure 17-8, Topple Failure
(Soil Slope and Embankment Design – September 2005)

17.3 LOADING CONDITIONS

The stability of slopes and ERSs is based on the height of the slope or ERS (i.e. the load) and the resistance of the subsurface soils (i.e. shear strength) to that loading. Increasing the height and steepness of the slope increases the potential for instability. It is incumbent upon the geotechnical engineer to know and understand the loading conditions for which the stability analysis is being performed to evaluate. All of these loading conditions apply to both natural and man-made fill and cut slopes, but each condition does not have to be analyzed in every case`. These loading conditions are listed below:

1. End-of-Construction (Short-term)
2. Long-term
3. Earthquake
4. Staged Construction
5. Rapid Drawdown
6. Surcharge Loading
7. Partial Submergence

Each of these loading conditions requires the selection of the appropriate soil strength parameters. Chapter 7 provides a more detailed discussion on the selection of drained and undrained soil shear strength parameters and the differences in total and effective stress. Once the rate of loading (i.e. loading condition) is determined, the soil response should be determined (i.e. drained or undrained). The drained response of soil is determined by loading the soil slowly enough to allow for the dissipation of pore pressures ($\Delta u = 0$). Conversely, the undrained response of a soil is determined by loading the soil faster than the pore pressures can dissipate ($\Delta u \neq 0$). This change in pore pressure can be either positive or negative depending on whether the soil compresses ($\Delta u > 0$) or dilates ($\Delta u < 0$). After determining the soil response (either drained or undrained), the type of analysis is selected based on the dissipation of pore pressures and the rate of loading. If the pore pressure increases with the application of load, i.e. during fast loading on a fine-grained soil, then a total stress analysis is conducted. If the loading does not produce a change in pore pressure, i.e. during slow loading of a fine-grained soil or the loading is placed on coarse-grained material, then an effective stress analysis is conducted.

According to Duncan and Wright (2005), "Whether slope stability analyses are performed for drained conditions or undrained conditions, the most basic requirement is that equilibrium must be satisfied in terms of total stress." In other words, all forces, including water that act on the slope or ERS, need to be accounted for in the stability analysis. The development of these forces allows for the determination of the total normal stress acting on the shear surface and the shear strength required to maintain equilibrium. Normal stresses are required to develop the soil shear strength ($\Phi > 0°$). The shear strength of cohesive, fine-grained ($\Phi = 0°$) is independent of the normal stress acting on the shear surface.

To develop effective stress shear strength parameters, the pore pressures along the shear surface need to be known and need to be subtracted from the total shear strength. For drained conditions, the pore pressures can be estimated using either hydrostatic or steady seepage boundary conditions. However, for undrained conditions, the pore pressures are a function of the response of the soil to shearing, therefore, the evaluation of the pore pressures is difficult. The development of total stress shear strength parameters does not require determination of pore pressures. Total stress analyses therefore can only be applied to undrained conditions. In total stress analyses, the pore pressures are determined as a function of the behavior of the soil during shear.

In drained soil response, the load is applied slow enough to allow for the dissipation of excess pore pressures. An effective stress analysis is performed using:

- Total unit weights
- Effective stress shear strength parameters
- Pore pressures determined from hydrostatic water levels or steady seepage analysis

Total unit weights are required in drained soil response. Since the majority of the analytical software packages account for the location of the groundwater table, it is incumbent on the engineer to know the requirements of the analytical software package and provide the correct input parameters.

In undrained soil response, the load is applied rapidly and excess pore pressures are allowed to build up. The pore pressures are controlled by the response of the soil to the application of the external load. A total stress analysis is performed using:

- Total unit weights
- Total stress shear strength parameters

The previous discussion dealt with the selection of total or effective stress strength parameters; however, these strength parameters are for peak shear strength. The use of peak shear strengths is appropriate for fill type slopes. However, the use of peak shear strength parameters in cut slopes should be considered questionable. Therefore, the use of residual shear strength shall be used in the design of cut slopes. Residual shear strength should be either determined from laboratory testing or using the procedures outlined in Chapter 7. The location of the water surface in cut slopes should be accounted for during design. The use of steady state seepage may be required, particularly, if the slopes intercepts the water table well above the toe of the slope. In addition, surface drainage features may be required to control the flow of groundwater as it exits the slope.

17.3.1 End-of-Construction Condition

The End-of-Construction condition also termed Short-term can have either drained or undrained soil response depending on the time for excess pore pressure ($\Delta u \neq 0$) dissipation. The time for excess pore pressure dissipation shall be determined using the method described in Chapter 7 or from consolidation testing of the embankment or ERS materials. If the time for pore pressure dissipation is determined to be days or weeks (typically cohesionless soils), then drained soil response should be used. Conversely, if the time for pore pressure dissipation is months to years (typically, plastic, fine-grained soils), then undrained soil response should be used. Engineering judgment should be used for the soils that have a time for pore pressure dissipation of weeks to months. The selection on the use of drained or undrained soil response should be based on the time for the completion of construction. In addition, the slope being analyzed may consist of materials that have both drained and undrained soil responses (i.e. the slope contains cohesionless and plastic fine-grained materials). The soil response of each layer should be determined based on the time for dissipation of pore pressures in each layer. The live load caused by traffic should be included in this analysis.

For the End-of-Construction loading condition for embankments and ERSs, the weight of the pavement and live load surcharges shall be applied. The loads shall be determined as specified in Chapter 8 of this manual. The load factor (γ_i) shall be taken as 1.0.

17.3.2 Long-term Condition

The Long-term condition should use a drained soil response model. The use of the drained soil response is based on the assumption that excess pore pressures have dissipated ($\Delta u = 0$). The time for dissipation of pore pressures should be determined, if the engineer suspects that not enough time has passed to allow for the dissipation. The appropriate soil response should be selected (i.e. drained if $\Delta u = 0$ or undrained if $\Delta u \neq 0$). During Long-term analysis, the live load surcharge (see Chapter 8) and the dead load induced by the existing pavement section and any asphalt overlays (see Chapter 8 and assume overlay has a thickness of 12 inches) should be included. Similar to the End-of-Construction loading condition, the load factor (γ_i) shall be 1.0.

17.3.3 Earthquake Condition

According to Duncan and Wright (2005), the stability of slopes and ERSs is affected by earthquakes in two ways; first the earthquake subjects the soils to cyclically varying loads and secondly, cyclic strains induced by these loads may lead to a reduction in the shear strength of the soil. The soil response during cyclic loading is undrained, since the load is applied rapidly and excess pore pressures do not have time to dissipate. In soils where the shear strength reduction is less than fifteen percent, a psuedostatic analysis is normally conducted. If the soil shear strength is reduced more than fifteen percent, then a dynamic analysis should be performed. See Chapter 7 for aid in determining the reduced shear strengths that should be used. The earthquake condition is discussed in greater detail in Chapter 13. In the earthquake analysis, include the dead load induced by the addition of asphalt to the pavement section, but do not include the live load surcharges.

17.3.4 Staged Construction Condition

The placement of embankments or ERSs over soft fine-grained soils may require vertical staging of the construction to prevent instability. The instability is caused by the low shear strength of the fine-grained soil and the increase in excess pore pressures. The increase in pore pressures lowers the drained shear strength of the soil; however, with the dissipation of the excess pore pressures, the effective strength of the soil will increase. Therefore, vertical staging can be used to increase the soil shear strength by placing a stage (thickness) of soil over the soft fine-grained soil and allowing the excess pore pressures to dissipate (waiting period). Once the excess pore pressures have dissipated, then the next stage can be placed. This process can be repeated until the final height of the embankment or ERS is achieved. The completion of placement of each stage is critical, since this is when instability is most likely to occur (i.e. the soil shear strength has not increased). Settlement of the stage will also occur as the excess pore pressures dissipate. Therefore, consolidation testing is required to determine the time rate of settlement, as well as the magnitude of total settlement, that is anticipated for each stage as well as the full embankment or ERS. If the time rate of settlement indicates that the waiting periods are too long, then wick drains may be used to increase the time to complete settlement. The increase in shear strength is a function of the Degree of Consolidation (U). Provided below are the equations relating the increase in undrained shear strength to U.

$$\Delta \mathbf{C_u} = \mathbf{U_t} * (\Delta\sigma * \tan(\varphi_{cu})) \qquad \text{Equation 17-1}$$

or

$$\Delta \mathbf{C_u} = \mathbf{U_t} * (\Delta\sigma * (\tau / \sigma'_{vo})) \qquad \text{Equation 17-2}$$

Where,

$$\tan(\varphi_{cu}) = 0.23(OCR)^{0.8} = \frac{\tau}{\sigma'_{vo}} \qquad \text{Equation 17-3}$$

Where,

ΔC_u = Increase in undrained cohesion
U_t = Degree of consolidation at a specific time (enter as decimal)
$\Delta\sigma$ = Increase in applied vertical stress
φ_{cu} = Consolidated undrained internal friction angle
τ = Undrained Shear Strength
σ'_{vo} = Vertical effective overburden stress
OCR = Overconsolidation Ratio (see Chapter 7)

The soil shear strength and consolidation parameters should be determined in accordance with the procedures outlined in Chapter 7. See Chapter 19 for information concerning prefabricated vertical drain design. The stability of the slope or ERS should be monitored using the instrumentation described in Chapter 25.

17.3.5 Rapid Drawdown Condition

The Rapid Drawdown Condition occurs when a slope or ERS that is used to retain water experiences a rapid (sudden) lowering of the water level and the internal pore pressures in the slope or ERS cannot reduce fast enough. For SCDOT projects, this is not a normal condition because SCDOT slopes or ERSs are not designed to retain water. However, in some situations water may build up against a slope or ERS, where this has occurred, Rapid Drawdown should be considered. For procedures on how to conduct rapid drawdown analysis, see Duncan and Wright, 2005.

17.3.6 Surcharge Loading Condition

Surcharge loads can be either temporary or permanent. Temporary surcharge loads can include construction equipment or additional fill materials placed on an embankment to increase the rate of settlement. Temporary soil surcharges are typically used in conjunction with staged constructed embankments. Therefore, the effects of the surcharge will need to be accounted for in staged construction. Typically, for temporary surcharges like equipment, the undrained shear strength of the soil should be used. Permanent surcharges consist of asphalt overlays. These permanent surcharge loads should be included in long term analysis.

17.3.7 Partial Submergence Condition

The partial submergence condition occurs when an embankment or ERS experiences the flood stage of a river or stream. When these conditions occur water, can penetrate the embankment or ERS and affect the shear strength of the soil. The amount of water that actually penetrates the embankment or ERS is a function of the permeability of the material used in the construction of the embankment or ERS. The permeability of the embankment material or retained backfill can be estimated as indicated in Chapter 7. Further, the duration of the flood event will also determine the effect of the flood on the embankment or ERS. The longer the flood lasts, the more the potential effect on the embankment or ERS.

17.4 LRFD SLOPE STABILITY

FHWA/AASHTO has recommended that the stability of an embankment slope or an ERS be determined using the Service limit state instead of the Strength limit state. Use of Service instead of Strength limit state accounts for two design issues. The first is that current slope stability analysis software does not allow for the input of load and resistance factors. Second is that most of the strength parameters, required in stability analysis, are derived from correlations (see Chapter 7). Further, the research is incomplete for the determination of resistance factors, since relatively few embankment or ERS failures occur, where the strength of the soil can be accurately determined and applied across a broad spectrum of soils. Therefore, the basic ASD calculation methods will continue to be used. After completion of the ASD analysis, the calculated Safety Factor (SF) is inversed to convert from ASD to LRFD.

$$\textbf{Driving Force } (\Sigma q_i) \le \textbf{Resistance Force } (\Sigma r_i) \qquad \text{Equation 17-4}$$

or

$$\mathbf{SF} = \frac{\sum \mathbf{r_i}}{\sum \mathbf{q_i}} \qquad \text{Equation 17-5}$$

As indicated in Chapter 8, the basic LRFD equation is

$$\mathbf{Q} = \sum \gamma_i \mathbf{Q_i} \le \varphi_n \mathbf{R_n} = \mathbf{R_r} \qquad \text{Equation 17-6}$$

Where,

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

In using the Service limit state versus the Strength limit state, the stability analysis reverts to the typical way of performing stability analysis since the various Service limit states (I, II, III, and IV) all use a load factor (γ_i) of 1.0. Therefore, Equation 17-5 can be rewritten:

$$\mathbf{Q} = \sum \mathbf{Q_i} \le \varphi \sum \mathbf{R_n} = \mathbf{R_r} \qquad \text{Equation 17-7}$$

or

$$\frac{1}{\varphi} = \frac{R_r}{Q} = \frac{\sum r_i}{\sum q_i} \qquad \text{Equation 17-8}$$

Equating Equation 17-5 with Equation 17-8 produces

$$\mathbf{SF} = \frac{\sum \mathbf{r_i}}{\sum \mathbf{q_i}} = \frac{1}{\varphi} \qquad \text{Equation 17-9}$$

Alternately Equation 17-9 may be written

$$\varphi = \frac{1}{\mathbf{SF}} \qquad \text{Equation 17-10}$$

Therefore, to obtain the required resistance factor (φ) from typical slope stability software packages, the SF obtained is simply inversed. The lower the resistance factor the higher the Safety Factor.

17.5 SLOPE STABILITY ANALYSIS

Stability analysis is based on the concept of limit equilibrium (i.e. Driving Force (Σq_i) ≤ Resistance Force (Σr_i)). The Driving Forces include the weight of the soil wedge (i.e. dead load), any live load surcharges and any other external loads (i.e. impact loads on ERSs). The Resisting Force is composed entirely of the shearing resistance of the soil. Limit equilibrium is defined as the state where the resisting force is just larger than the driving force (i.e. SF = 1.01 or φ = 0.99). According to Duncan and Wright (2005), the equilibrium can be determined for either "single free body or for individual vertical slices." Regardless of how equilibrium is determined, three static equilibrium conditions must be satisfied.

- Moment equilibrium
- Vertical equilibrium
- Horizontal equilibrium

Not all methods resolve all of the equilibrium conditions; some just resolve one condition while others solve two conditions and assume the third condition is zero. Other methods solve all three equilibrium conditions.

The single free body looks at the driving forces and the resisting forces along an assumed failure surface. These solutions tend to be relatively simple and are more conducive to the use of design charts. The use of design charts is applicable to preliminary design and should therefore be used when conducting preliminary design. The second method of solving equilibrium is dividing the slope into individual vertical slices. There are numerous procedures that resolve equilibrium using vertical slices. Listed below are some of the more common types.

- Ordinary Method of Slices
- Simplified (Modified) Bishop
- Force Equilibrium
- Spencer
- Morgenstern and Price

The Ordinary Method of Slices and Simplified Bishop assume a circular failure surface while the others assume a non-circular failure surface. The Spencer and Morgenstern and Price both provide a solution for all three equilibrium conditions. The vast majority of slope stability software packages are capable of using more than one method to determine the stability of a slope or ERS, and changing the method will affect the resistance factor (φ = 1/SF). Therefore, the method that will produce the lowest Factor of Safety (highest resistance factor) shall be used. A minimum of three methods shall be used to determine slope stability with the Simplified (Modified) Bishop Method being required. The above list is not meant to be all inclusive. If other methods not found on the list are used, the Geotechnical Report shall include a detailed discussion of the methodology, including previous projects on which the method was used by the geotechnical engineer-of-record. The use of Factor of Safety recognizes the fact that virtually all software currently in use to determine the stability of slopes utilizes Allowable Strength Design (ASD) as opposed to LRFD. Using hand calculations is recommended to check the final solution provided by a software package provided the method used to determine stability can be readily solved by hand. A brief discussion of the methods listed above is provided in the following sections.

The use of computer generated solutions for slope stability should be limited to final designs only. The use of slope stability design charts is permitted for preliminary design during the preliminary exploration. The results of these charts are reported in the preliminary road geotechnical report. For detailed explanation and discussion on how to use the charts, see Appendix B.

17.5.1 Ordinary Method of Slices

The Ordinary Method of Slices (OMS) is one of the earliest methods for determining the stability of a slope and was developed by Fellenius in 1936. This method solves moment equilibrium conditions only and is applicable only to circular failure surfaces. This method does not solve either vertical or horizontal equilibrium conditions. This method is relatively simplistic and can be solved by hand easily. This method should not be used during final design. Its inclusion here is for completeness of the various slope stability methods.

17.5.2 Simplified Bishop

The Simplified Bishop method was developed by Bishop in 1955 and can also be called the Modified Bishop method. The Simplified Bishop method solves two of the equilibrium equations, moment and vertical. This method assumes that horizontal forces are not only perpendicular to the vertical sides of the slice, but are equal and opposite; therefore, the horizontal forces cancel out. Since horizontal equilibrium is not satisfied, the use of the Simplified Bishop method in pseudo-static seismic design is questionable and should therefore not be used. As with the OMS, Simplified Bishop can only be used on circular failure surfaces.

17.5.3 Force Equilibrium

For the Force Equilibrium method of determining slope stability, depending on the method selected (Lowe and Karafiath; Simplified Janbu; Modified Swedish), the moment equilibrium is either ignored or assumed to be zero. Duncan and Wright (2005) and the US Army Corps of Engineers (2003) contain a detailed description of each of these Force Equilibrium methods. The Force Equilibrium method solves both the horizontal and vertical forces. The main assumption required using this method is the inclination of the horizontal forces on the given slice. The inclination of the horizontal forces acting on slice may be either the slope angle or the average slope angle if multiple slopes are involved (i.e. a broken back slope). In addition, the Force Equilibrium methods solve non-circular failure surfaces and therefore, may be solved graphically.

17.5.4 Spencer's Method

Spencer's Method solves all three conditions of equilibrium and is therefore termed a complete limit equilibrium method. Spencer's Method was originally developed to determine the stability of circular failure surfaces; however, Wright (1969) determined that the Spencer Method could also be used on non-circular failure surfaces as well. This method assumes that the interslice forces are parallel and act on an angle (θ) above the horizontal. This angle is one of the unknowns in this method. Therefore, a first approximation of the angle should be the slope angle. The other unknown is the Factor of Safety. Because the method solves for Factor of Safety, an iterative process is required; therefore, this method lends well to the use of computers.

17.5.5 Morgenstern and Price Method

The Morgenstern and Price Method is very similar to the Spencer Method. The main difference between the two methods is that Spencer solves for the interslice angle, while the Morgenstern and Price Method solves for the scaling parameter that is used on a function that describes the slice boundary conditions. The Morgenstern and Price Method provides added flexibility using the interslice angle assumptions.

17.6 SETTLEMENT – GENERAL

Regardless of the type of structure, embankments, ERSs, bridges or buildings are all placed on geomaterials (i.e. soil and rock) and will therefore potentially undergo settlement. According to FHWA-NHI-05-132, *Soil Slope and Embankment Design*, settlement is comprised of three components: immediate (elastic or instantaneous), primary consolidation and secondary compression. Settlements (strains) are caused by an increase in the overburden stress (i.e. increase in load or demand). In many cases, the amount of settlement (strain) determines the capacity (resistance) to a load (demand).

$$\Delta_V = S_t = S_i + S_c + S_s \qquad \text{Equation 17-11}$$

Where,

S_t = Δ_V = Total Settlement
S_i = Immediate Settlement
S_c = Primary Consolidation Settlement
S_s = Secondary Compression Settlement

Since immediate settlement (S_i) is typically built out during construction, the total settlement should be reported as the amount of primary consolidation (S_c) and secondary compression (S_s) settlement for use in comparing to the Performance Limits established in Chapter 10. However, the effect of the immediate settlement should be determined, (i.e., will the immediate settlement affect the existing embankment or bridge structure, or will the borrow excavation quantities need to be increased). In addition, to determining the total amount of settlement that geomaterials will undergo, the time for the settlement should also be determined. From the combination of total settlement and time for settlement to occur, a rate can be determined and compared to the Performance Limits established in Chapter 10. Another phenomenon that occurs in very soft fine-grained soils is lateral squeeze. Lateral squeeze can cause both vertical as well as lateral displacements. These displacements may induce loadings on structures that have foundations located in the layer susceptible to squeeze.

17.7 CHANGE IN STRESS

As indicated previously, in order for settlement to occur there must be an increase in stress placed on the geomaterials, especially in the case of soil. The increase in stress can be caused by placement of an embankment, shallow or deep foundation or dewatering. The effects of dewatering will not be described in this Manual; however, should dewatering be required, an expert in dewatering should be consulted. There are various methods for determining the change in stress at different depths within the soil profile. The method used in this Manual is the Boussinesq method. Discussed below is the change in stress caused by shallow foundations and by the placement of an embankment. In addition, the increased stress on buried structures caused by the placement of fill is also discussed.

17.7.1 Shallow Foundations

Shallow foundations, as indicated in Chapter 15, are used to support bridges, ERSs and other ancillary transportation facilities. Earthen embankments are theoretically supported by shallow foundations; however, the change in stress caused by embankments is discussed in the following section. According to Chapter 15, a spread footing has a length to width (L_f/B_f) ratio of less than five. Shallow foundations having a length to width ratio greater than five are termed strip or continuous footings. Figure 17-9 depicts the approximate distribution of stresses beneath spread (square) and continuous footings.

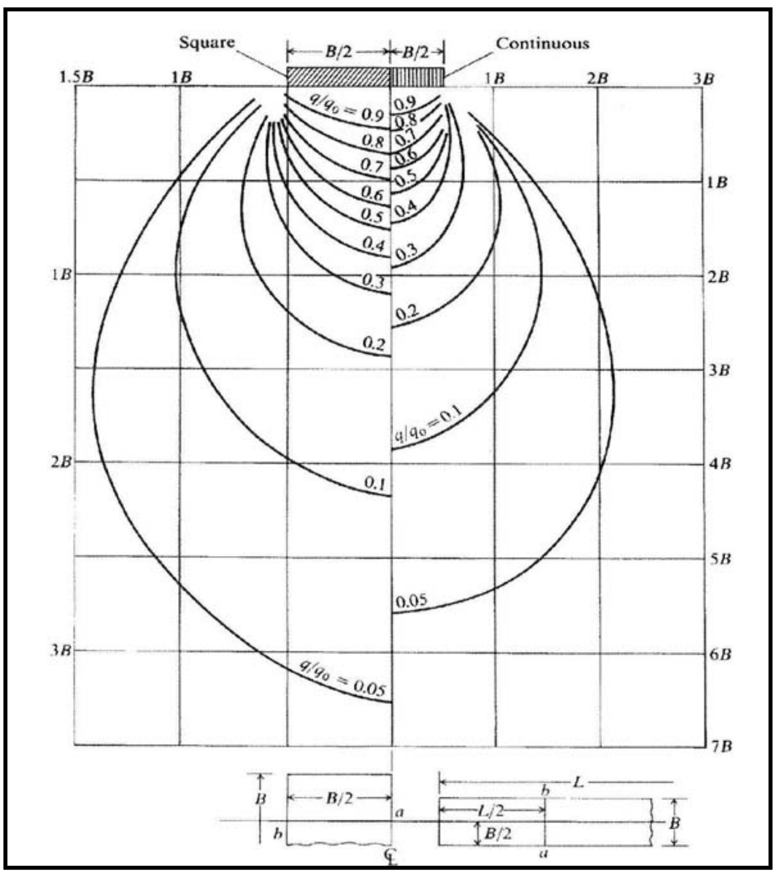


Figure 17-9, Stress Isobars
(Foundation Analysis and Design - 1986)

Where,

B = Foundation width
L = Foundation length
q = Stress at depth indicated
q_o = Applied vertical stress

Figure 17-9 should only be used as an approximate estimate of the increase in stress on a soil layer. To more accurately determine the increase in stress caused by a shallow foundation, the following equation should be used.

$$\Delta\sigma_v = \int_0^r \frac{3q_o}{2\pi Z^2} \frac{1}{\left[1+\left(\frac{r}{Z}\right)^2\right]^{\frac{5}{2}}} 2\pi dr \qquad \text{Equation 17-12}$$

Where,

q_o = Applied vertical stress
Z = Vertical depth below load
r = Horizontal distance between the load application and the point where the vertical pressure is being determined

Newmark (1935) performed the integration of this equation and derived an equation for the increase in vertical stress beneath a corner of a uniformly loaded area.

$$\Delta\sigma_v = Iq_o \qquad \text{Equation 17-13}$$

Where,

I = Influence factor which depends on m and n
m = x/z
n = y/z
x = Width of the loaded area
y = Length of loaded area
z = Depth below the loaded surface to the point of interest
q_o = Applied vertical stress

The influence factor, I, can be obtained from Figure 17-10.

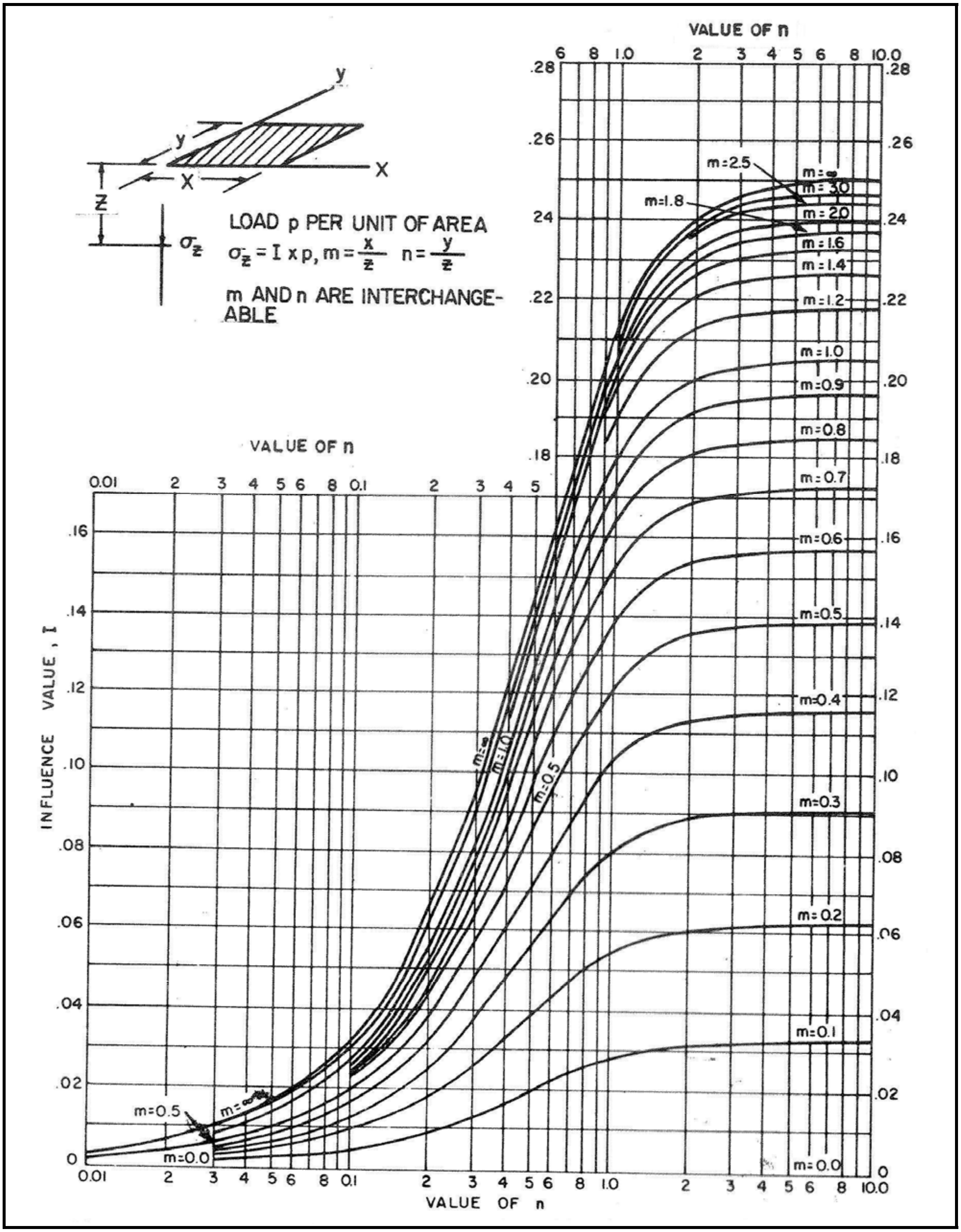


Figure 17-10, Influence Factor Chart
(NAVFAC DM 7.1 - 1982)

The change in stress determined using Equation 17-13 is for a point underneath the corner of a loaded area. Therefore, the change in stress at a depth underneath the center of the footing is four times the amount determined from Equation 17-13. The change in stress at the mid-point of an edge of a footing is twice the amount determined from Equation 17-13. To find the change in stress at points other than the middle, middle of the edge or a corner of a footing, the Principle of Superposition is used. The Principle of Superposition states that the change in stress at any point is sum of the stresses of the corners over the point (see Figure 17-11).

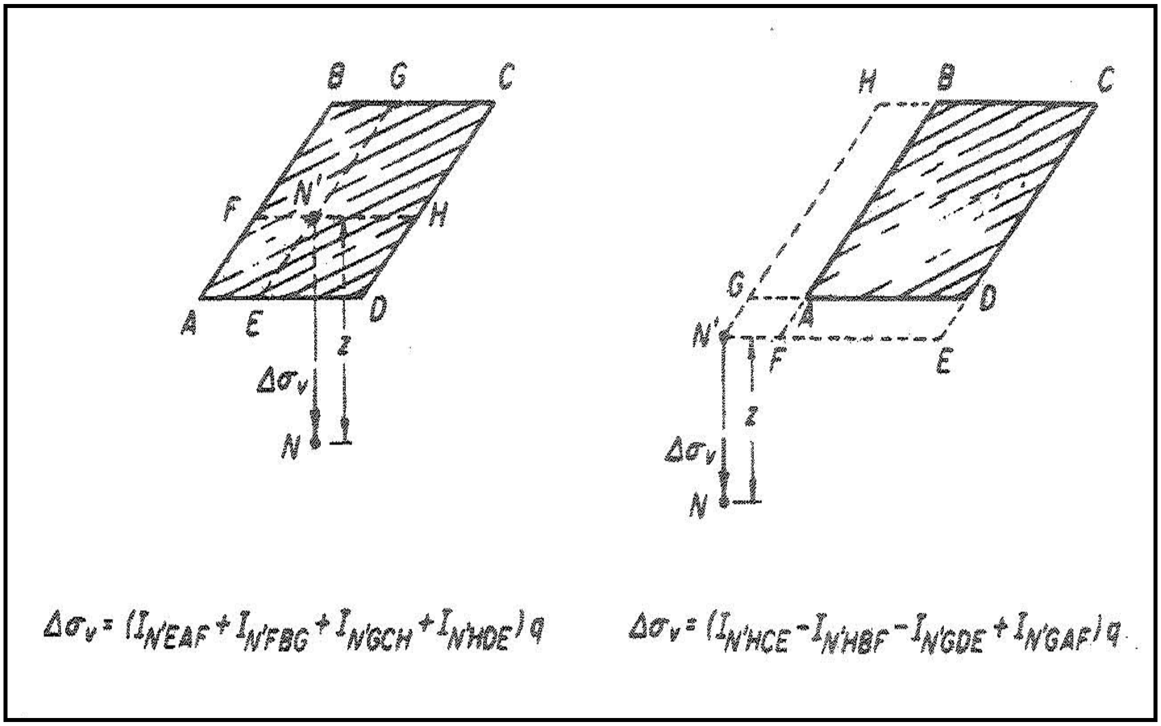


Figure 17-11, Principle of Superposition
(Soil Slope and Embankment Design – September 2005)

17.7.2 Embankments

The change in stress beneath embankments is determined differently than for shallow foundations, because the area loaded by an embankment is larger than for a shallow foundation. Further the change in stress beneath an embankment is complicated by the geometry of the embankment, i.e. the sides slope downward. Embankments comprise two groups; those with infinite length (i.e. side slopes) and those with finite length (i.e. end slopes). The first group is generally longitudinal to the direction of travel, while the other is generally transverse to the direction of travel. For infinite slope embankments, the loading can be represented as a trapezoid. The change in stress beneath an embankment is determined using Equation 17-13. Osterberg (1957) integrated the Boussinesq equations to develop the influence factors (I). Figure 17-12 provides the chart for determining the influence factors (I) for use in Equation 17-13.

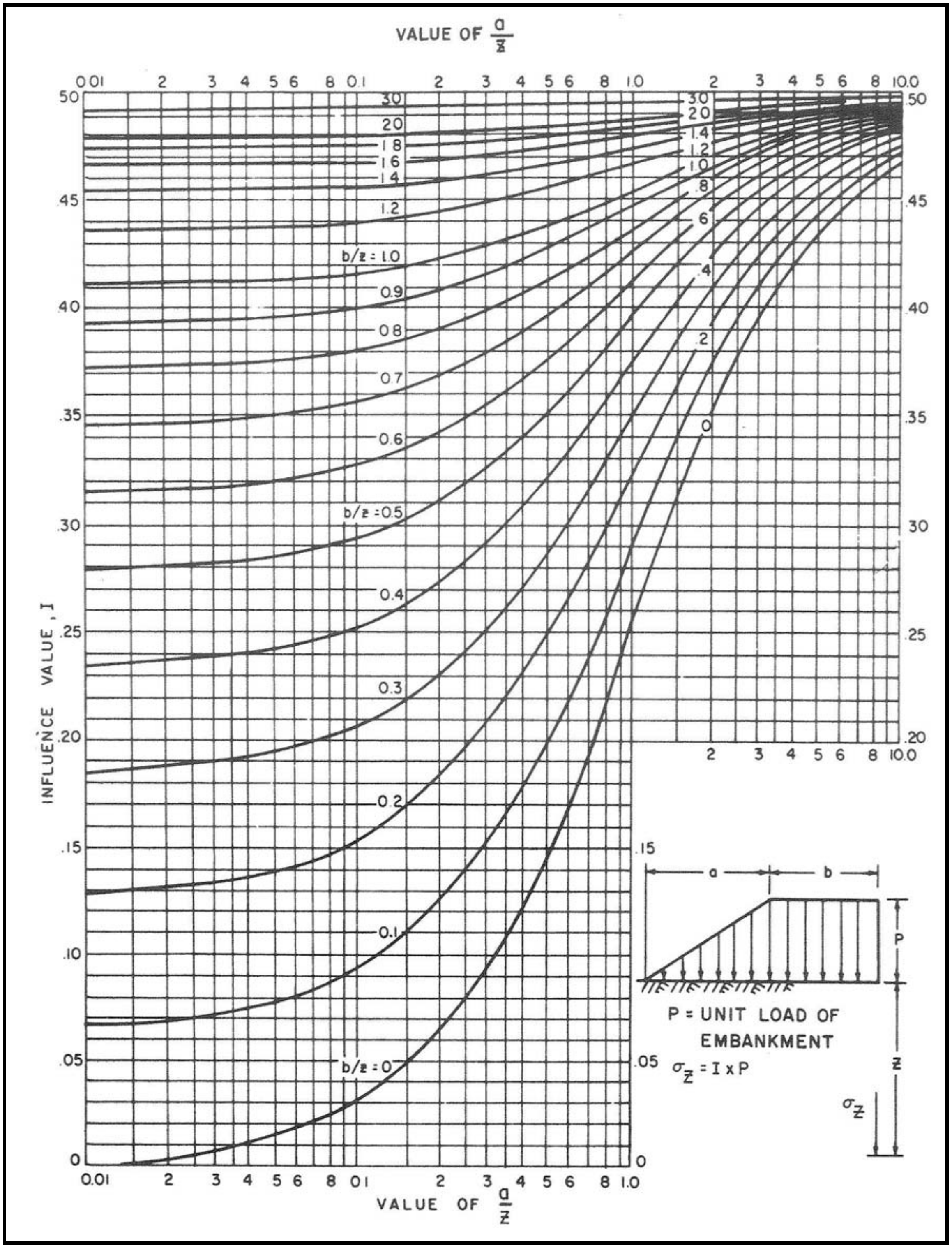


Figure 17-12, Influence Factor Chart – Infinitely Long Embankments (NAVFAC DM 7.1 - 1982)

For finite slopes, Equation 17-13 is used to determine the change in stress. However, the development of the influence factor (I) is complicated by the geometric requirements of the slope. The loading consists of two components; first the area load of the embankment and second the load of the sloping section. The influence factor for the area load is determined using Figure 17-10. Note that the stress is doubled, because the stress is determined at a corner of the loaded area. The influence factor for the sloping portion is determined from Figures 17-13 and 17-14.

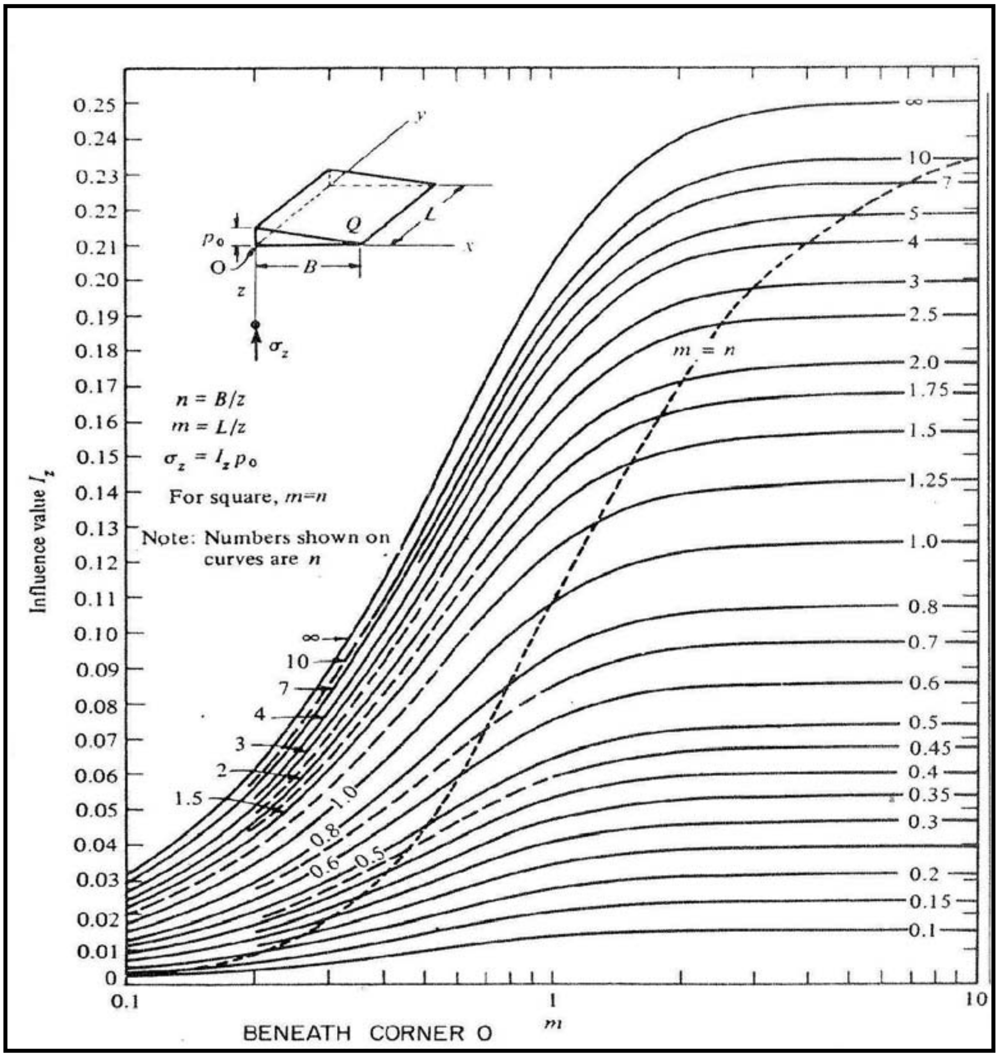


Figure 17-13, Influence Chart Beneath Crest of Slope (NAVFAC DM 7.1 - 1982)

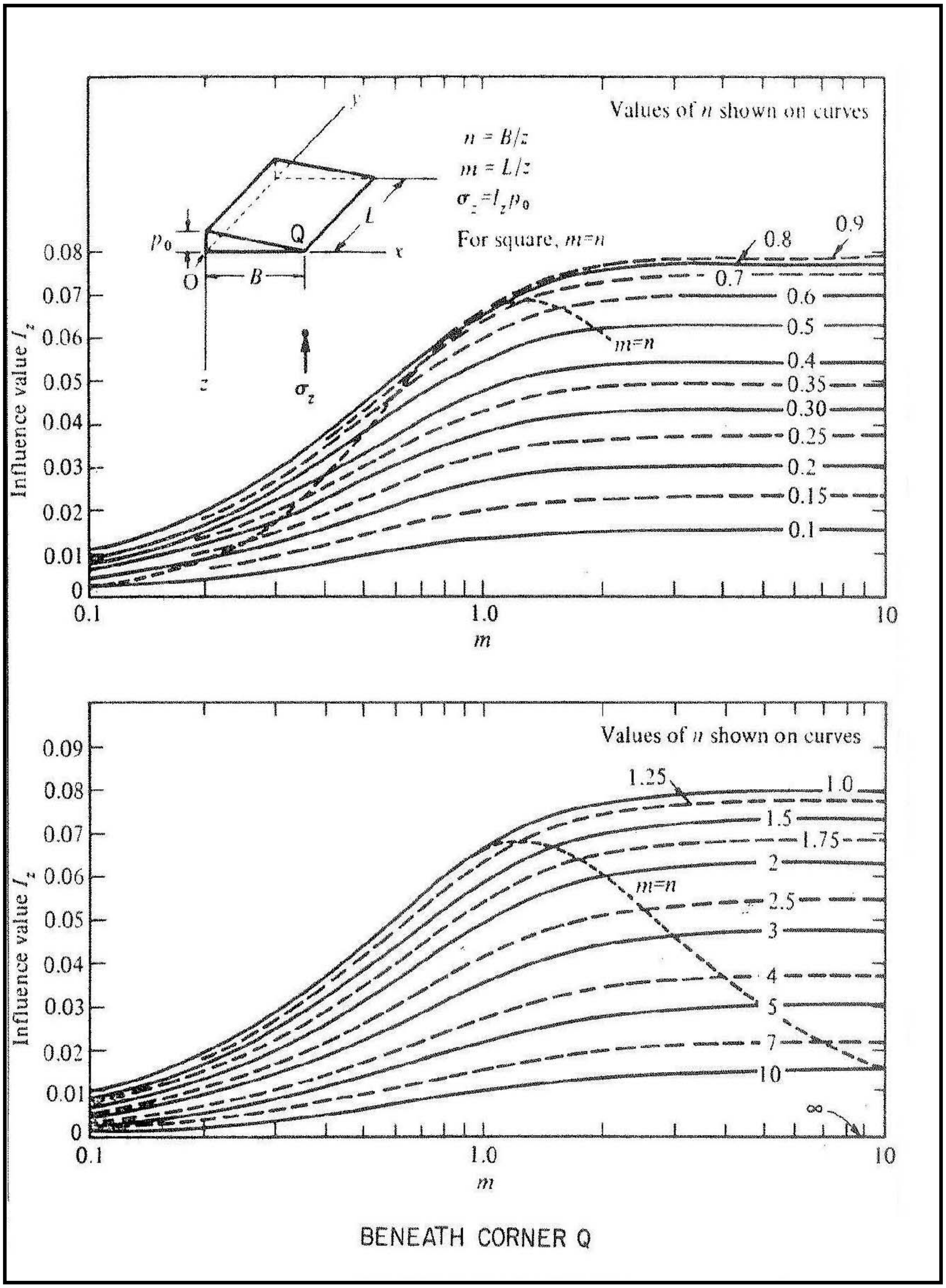


Figure 17-14, Influence Chart Beneath Toe of Slope (NAVFAC DM 7.1 - 1982)

As alternate to the procedures indicated above for finding the change in stress between a sloped embankment, the procedure described in the *Soils and Foundations Workshop Manual* (July 2000) may be used. This method was originally developed by the New York State Department of Transportation. This method uses the following equation.

$$\rho = \boldsymbol{K}\gamma_{\boldsymbol{f}}\boldsymbol{h} \qquad \text{Equation 17-14}$$

Where,

ρ = Change in vertical stress caused by the embankment ($\Delta\sigma_v$)
K = Influence factor from Figure 17-15
γ_f = Unit Weight of fill Material
h = Height of embankment (see Figure 17-15)

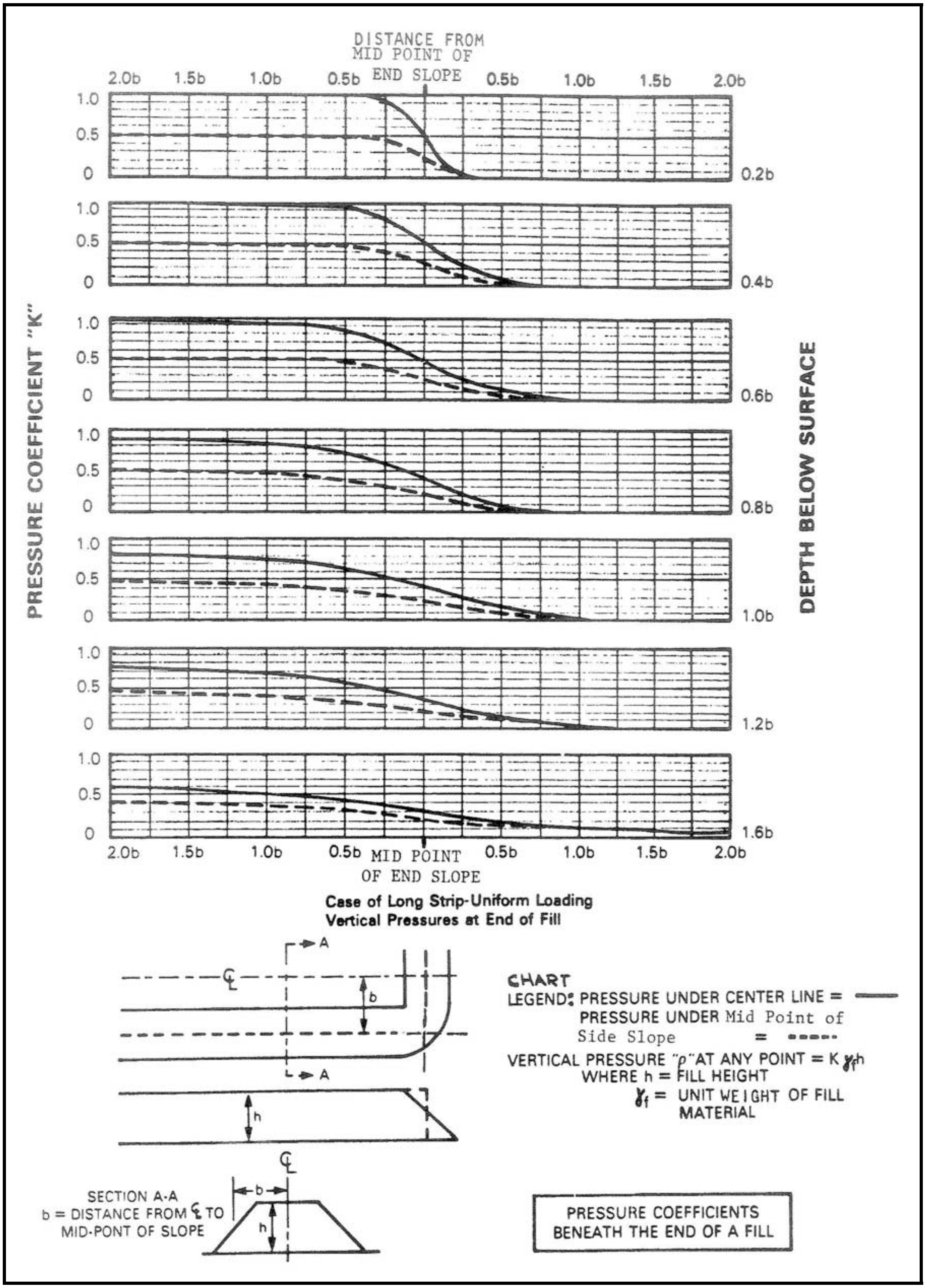


Figure 17-15, Pressure Coefficients Beneath the End of a Fill (Soil and Foundations Workshop Manual – November 1982)

17.7.3 Buried Structures

Buried structures consist of culverts, pipes, boxes, etc. and are required to be designed to handle the loads imposed by embankments. The structural design of these structures is beyond the scope of this Manual; however, the design of buried structures is handled in AASHTO LRFD Bridge Design Specifications (latest edition), Section 12 – Buried Structures and Tunnel Liners. One of the parameters required is the unit weight of the fill material above the buried structure. Therefore, the unit weight (γ_f) may be required to satisfy design requirements along with the other required fill parameters.

17.8 IMMEDIATE SETTLEMENT

Immediate settlement, also termed elastic or instantaneous, occurs upon initial loading of the subgrade soils. This type of settlement occurs in both cohesionless and cohesive soils. The amount of settlement is based on elastic compression of the soils and the time for settlement to occur typically ranges in the days to months (one to three) or typically during construction. The settlement consists of the compression of air filled voids (cohesive soils) and the rearrangement of soil particles (cohesionless soils).

17.8.1 Cohesionless Soils

Cohesionless soils as defined in Chapter 7 consist of sands, gravels, low plasticity silts and residual soils. In cohesionless soils, the immediate (elastic) settlement typically comprises the total amount of settlement anticipated. This Section provides several different methods for determining the immediate settlement of cohesionless soils. All of the methods shall be used and the largest settlement shall be used to compare to the performance limits provided in Chapter 10. Three of the methods are based on the Standard Penetration Test (SPT), one on the Electro-piezocone Test (CPT) and one on the Dilatometer Test (DMT).

17.8.1.1 SPT Methods

As indicated previously, there are three SPT methods for determining immediate settlement of cohesionless soils. The first method is the Hough (1959) method and is used in AASHTO Specifications (Section 10.6 – Spread Footings). The amount of immediate settlement (S_i) is determined using the following equation.

$$S_i = \sum_{i=1}^{n} \left(\frac{1}{C'}\right) H_i \log\left(\frac{\sigma'_{vo} + \Delta\sigma'_v}{\sigma'_{vo}}\right) \qquad \text{Equation 17-15}$$

Where,

C' = Bearing Capacity Index (from Figure 17 -16)

H_i = Thickness of the ith layer

σ'_{vo} = Effective overburden pressure at the mid-point of ith layer

$\Delta\sigma'_v$ = Change in effective vertical stress at the mid-point of ith layer

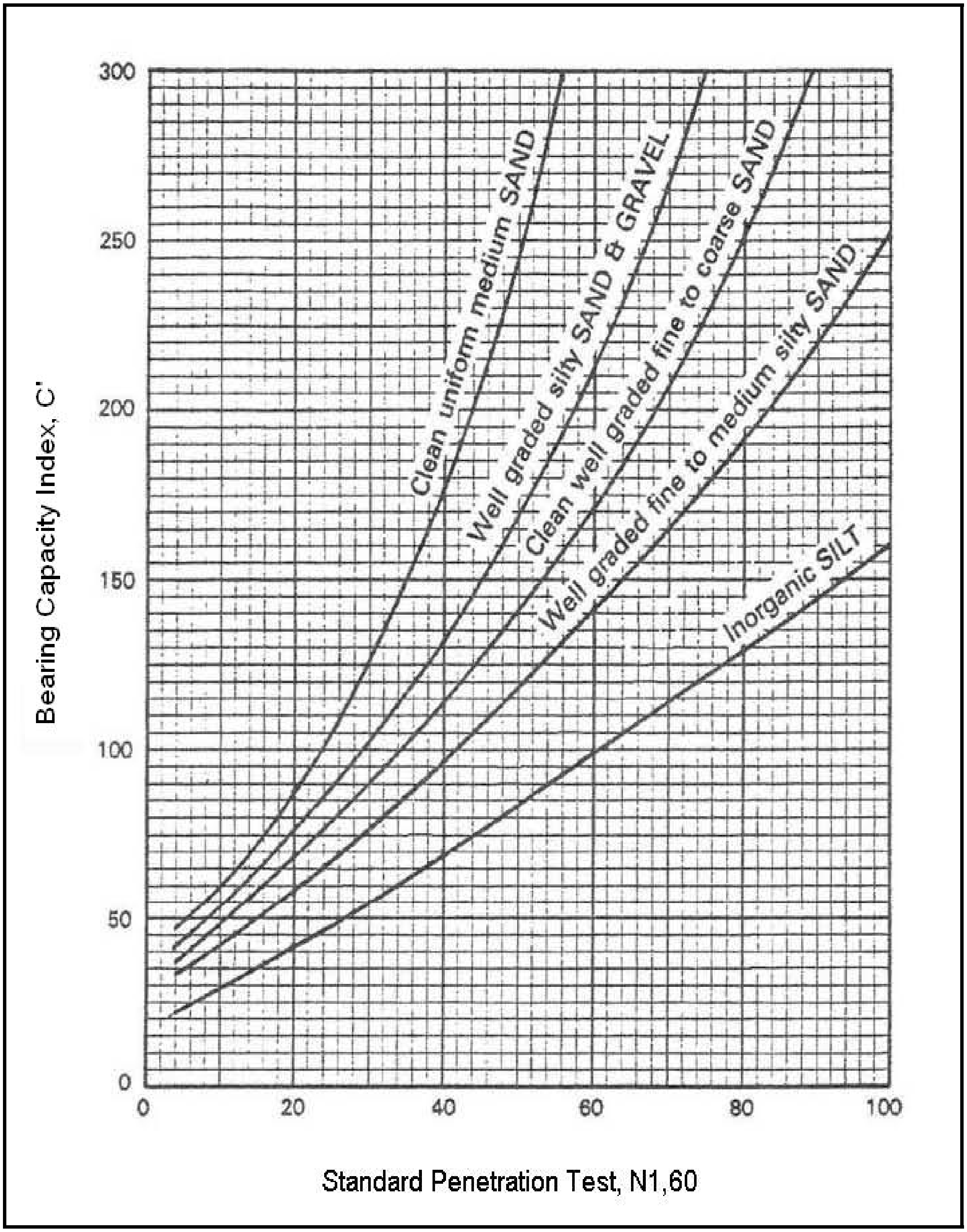


Figure 17-16, Bearing Capacity Index Chart
(Soil Slope and Embankment Design – September 2005,
Modified from Hough, 1959)

The $N_{1,60}$ is determined using the methodology discussed in Chapter 7.

The second SPT method of determining elastic settlement is the Peck and Bazaraa Method (Shallow Foundations (2001)). This method is a modification of the method described by Terzaghi and Peck (1967). It should be noted the equation used to determine the settlement is in SI units. The Peck and Bazaraa Method equation is listed below.

$$S_i = 0.265 C_w C_D \left(\frac{2q_o}{(N_{1,60})_{ave}} \right) \left(\frac{2B_f}{B_f + 0.3} \right)^2 \qquad \text{Equation 17-16}$$

$$C_w = \left(\frac{\sigma_{vo}}{\sigma'_{vo}} \right) \qquad \text{Equation 17-17}$$

$$C_D = 1 - 0.4 \left(\sqrt{\frac{\gamma D_f}{q_o}} \right) \qquad \text{Equation 17-18}$$

Where,

S_i = Immediate settlement in millimeters (mm) [1 mm = 0.03937 in]
C_w = Water table correction factor, at a depth of one-half of B_f
C_D = Embedment correction factor, use 1.0 when filling above original grade
B_f = Footing width in meters (m) [1 m = 3.28084 ft]
D_f = Depth of footing base embedment from ground surface in meters
γ = Total unit weight of soil in kiloNewtons per cubic meter (kN/m^3)
[1 kN/m^3 = 6.3656 lbs/ft^3)
q_o = Applied vertical stress or bearing pressure in kiloPascals (kPa) [1 kPa = 0.0209 ksf]
σ_{vo} = Total overburden pressure
σ'_{vo} = Effective overburden pressure
$(N_{1,60})_{ave}$ = Average $N_{1,60}$-value from base of footing to a depth of B_f below footing

The third SPT method was developed by Duncan and Buchignani (1976) based on Meyerhof (1965). The immediate settlement equation is provided below.

$$S_i = \frac{5q_o}{(N_{60} - 1.5)C_B} \qquad \text{Equation 17-19}$$

Where,

q_o = Applied vertical stress in tsf
N_{60} = Standard Penetration value corrected only for energy (see Chapter 7)
C_B = Width Correction (see Table 17-1)

The N_{60}-value is an average value from the base of the footing to a depth of B_f.

Table 17-1, Width Correction Factor, C_B (Duncan and Buchignani, 1976)

Footing Width, B (feet)	C_B
≤ 4	1.00
6	0.95
8	0.90
10	0.85
≥ 12	0.80

Duncan and Buchignani (1976) indicate that immediate (elastic) settlement will continue to increase over time (i.e. creep). The total amount of elastic settlement over time is determined using the following equation.

$$S_{iet} = S_i C_t \qquad \text{Equation 17-20}$$

Where

S_{iet} = Elastic settlement after a period of time

S_i = Immediate or elastic settlement

C_t = Time rate factor (see Table 17-2)

Table 17-2, Time Rate Factors (Duncan and Buchignani, 1976)

Time	C_t
1 month	1.0
4 months	1.1
1 year	1.2
3 years	1.3
10 years	1.4
30 years	1.5

For times other than those depicted in Table 17-2, the following equation may be used.

$$C_t = 0.0858Ln(t) + 0.9907 \qquad \text{Equation 17-21}$$

Where,

C_t = Time rate factor

t = Time period over which settlement occurs in months

17.8.1.2 CPT Method

There is one CPT method available for determining the immediate settlement of cohesionless soils. It was developed by Schmertmann (1970) and is applicable only to shallow foundations (i.e., a rigid structure is required). The Schmertmann Method uses the following equations.

$$S_i = C_D C_t q_{net} \sum_{i=1}^{n} \left(\frac{H_i}{E_{si}} \right) I_{azi} \qquad \text{Equation 17-22}$$

$$C_D = 1 - 0.5\left(\frac{\sigma'_D}{q_{net}}\right) \geq 0.5 \qquad \text{Equation 17-23}$$

$$q_{net} = q_o - \sigma'_D \qquad \text{Equation 17-24}$$

$$C_t = 1 + 0.2\log\frac{t}{0.1} \qquad \text{Equation 17-25}$$

$$I_P = 0.5 + 0.1\sqrt{\frac{q_{net}}{\sigma'_{Ip}}} \qquad \text{Equation 17-26}$$

Where,

C_D = Depth correction factor
C_t = Creep correction factor (t > 0.1 years)
q_{net} = Net total average bearing pressure in kPa
H_i = Thickness of the i^{th} layer in meters
E_{si} = Modulus of Elasticity of the i^{th} layer in kPa
I_{azi} = Average vertical strain influence factor of the i^{th} layer (from Figure 17-17)
q_o = Applied bearing pressure in kPa
σ'_D = Vertical effective stress at the base of the footing in kPa
t = time in years
σ'_{Ip} = Vertical effective stress at the depth of the peak influence factor (I_P) in kPa

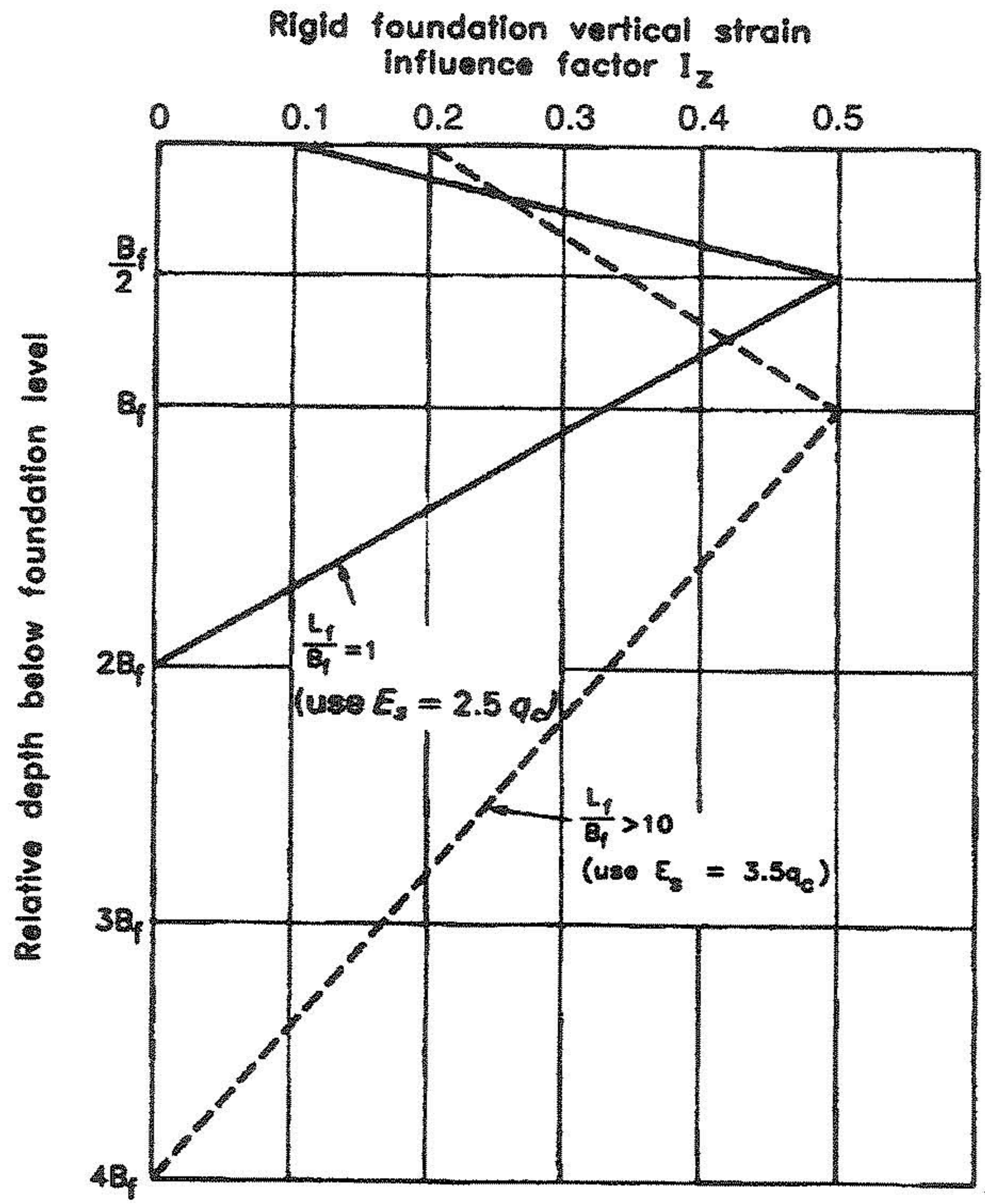


Figure 17-17, Vertical Strain Influence Factor Chart (Shallow Foundations – June 2001)

The modulus of elasticity (E_s) can be developed directly from laboratory testing, from Chapter 7 or from the following equation. This equation applies only to determining the E_s from CPT data.

$$E_s = F_s q_c \qquad \text{Equation 17-27}$$

Where,

F_s = Correlation factor depending on cone and soil type, stress level and footing shape (see Table 17-3)

q_c = Cone penetration tip resistance in kPa

Table 17-3, Correlation Factor, F_s
(Shallow Foundations – June 2001)

Case	F_s
$L_f/B_f = 1$	2.5
$L_f/B_f > 10$	3.5

Where,

B_f = Footing width

L_f = Footing length

As an alternate to Figure 17-17, the vertical strain influence factor may be determined using the equations in Table 17-4.

Table 17-4, Vertical Strain Influence Factor Equations
(Shallow Foundations – June 2001)

Footing Shape	I Terms	H_i	I_{azi} Equations
$L_f/B_f < 1$ (square)	I_{zsq}	0 to $B_f/2$	$\mathbf{0.1} + \left(\frac{H_i}{B_f}\right)(2I_P - 0.2)$
		$B_f/2$ to $2B_f$	$\mathbf{0.667}I_P\left(2 - \frac{H_i}{B_f}\right)$
$L_f/B_f > 10$ (continuous)	I_{zc}	0 to B_f	$\mathbf{0.2} + \left(\frac{H_i}{B_f}\right)(I_P - 0.2)$
		B_f to $4B_f$	$\mathbf{0.333}I_P\left(4 - \frac{H_i}{B_f}\right)$
$1 < L_f/B_f < 10$ (rectangular)	I_{zr}	N/A	$I_{zsq} + 0.111(I_{zc} - I_{zsq})\left(\left(\frac{L_f}{B_f}\right) - 1\right)$

17.8.1.3 DMT Method

Immediate settlement can be determined from Dilatometer Test data. The method is described in detail in *The Flat Dilatometer Test* published by the Federal Highway Administration. The equation for determining immediate settlement is provided below.

$$S_i = \sum_{i=1}^{n} \left(\frac{\Delta\sigma_v}{M_i} \right) H_i \qquad \text{Equation 17-28}$$

Where,

$\Delta\sigma_v$ = Change in vertical stress at the mid-point of i^{th} layer
M_i = Averaged constrained modulus of the i^{th} layer
H_i = Thickness of the i^{th} layer

17.8.2 Cohesive Soils

There is some immediate settlement in cohesive (clays and plastic silts) soils with most settlement occurring within a relatively short period after loading is applied. Most of this immediate settlement is from distortion and compression of air filled voids. It is anticipated that very little immediate settlement would occur in saturated cohesive soils. However, for unsaturated and/or highly overconsolidated (OCR $\geq$ 4) cohesive soils immediate settlement can be a predominant portion of the total settlement (S_t). These immediate settlements can be determined using the Theory of Elasticity and are determined using the following equation.

$$S_i = \frac{\left[q_o \left(1 - \nu^2\right) \sqrt{A} \right]}{\frac{E_s}{\beta_z}} \qquad \text{Equation 17-29}$$

Where,

q_o = Applied bearing pressure at the bottom of loaded area
ν = Poisson's ratio (see Chapter 7)
A = Contact area of the load
E_s = Modulus of elasticity for the soil (see Chapter 7)
β_z = Footing shape and rigidity factor (see Table 17-5)

Table 17-5, Footing Shape and Rigidity Factors (Shallow Foundations – June 2001)

L_f/B_f	β_z Flexible	β_z Rigid
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

Where B_f and L_f are the same as in the CPT Method described previously. If L_f/B_f is between five and ten, use linear interpretation. For L_f/B_f greater than ten, use the β_z for ten. The elastic parameters Poisson's ratio (ν) and modulus of elasticity (E_s) are provided in Chapter 7 for cohesive soils.

Christian and Carter (1978) developed an improved Janbu approximation for determining immediate settlement in cohesive soils. The improved Janbu approximation is provided below and assumes that the Poisson's ratio (ν) of the soil is 0.5.

$$S_i = \mu_0 \mu_1 \left(\frac{q_o B_f}{E_s} \right) \qquad \text{Equation 17-30}$$

Where,

- μ_0 = Influence factor for depth (see Figure 17-18)
- μ_1 = Influence factor for foundation shape (see Figure 17-18)
- q_0 = Applied vertical stress at the bottom of loaded area
- B_f = Foundation width
- L_f = Foundation length
- D = Foundation depth below ground surface
- H = Distance between bottom of foundation and a firm (non-compressible) layer
- E_s = Modulus of elasticity for the soil (see Chapter 7)

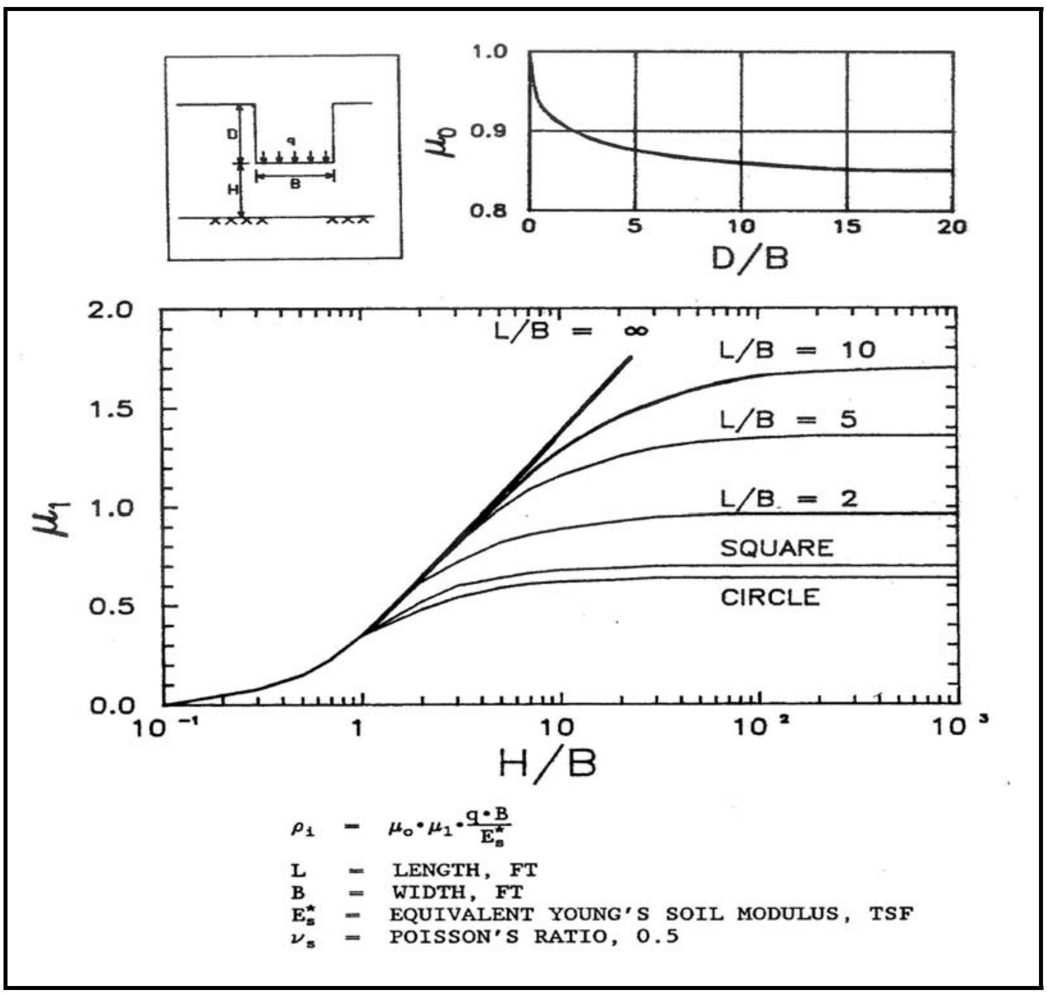


Figure 17-18, Janbu Influence Factor Chart (Settlement Analysis - 1990)

17.9 PRIMARY CONSOLIDATION SETTLEMENT

Primary consolidation settlement (S_c) occurs when the increase in load on a soil results in the generation of excess pore pressures within the soil voids. Depending on the type of soil, the time to reduce the excess pore pressures to some steady state level may be rapid (cohesionless soils) or may require long periods of time (cohesive soils). Therefore, primary consolidation settlement is comprised of two components, the amount of settlement and the time for settlement to occur. The amount of time required for cohesionless soil to settle is relatively short, typically occurring during construction, and the amount of settlement can be determined using immediate or elastic settlement theory. Therefore, the remainder of this Section will exclusively look at cohesive (clay and plastic silt) soils. Typically, normally consolidated (OCR = 1) soils undergo primary consolidation settlement. For the purpose of this Chapter, all plastic cohesive soils that have an OCR of less than four shall have the primary consolidation settlement determined.

The determination of primary consolidation settlement is based on six steps as presented in Table 17-6.

Table 17-6, Primary Consolidation Settlement Steps (Modified from Shallow Foundations – June 2001)

Step	Item
1	Computation of the initial vertical effective stress (σ'_{vo}) [total vertical stress (σ_{vo}) and pore water pressure (u_o)] of the layer(s) midpoint
2	Determination of preconsolidation stresses (σ'_p or p'_c)
3	Computation of changes in vertical effective stress ($\Delta\sigma'_v$) [associated with changes in both total stress ($\Delta\sigma_v$) and pore water pressures (Δu)] due to the construction
4	Determination of compressibility of the clay or plastic silt
5	Computation of layer compressions (S_c)
6	Computation of time for compressions

17.9.1 <u>Amount of Settlement</u>

In cohesive soils, loads are carried first by the pore water located in the interstitial space between the soil grains and then by the soil grains. The pore water pressure increases proportional to the load applied at that depth. As the excess pore water pressures reduce through drainage, the load is transferred to the soil grains. This drainage causes the settlement of cohesive soils. Therefore, the settlement is directly proportional to the volume of water drained from the soil layer. Typically, the area loaded is large, resulting in the flow of water vertically (either up or down) and not horizontally. Therefore, one-dimensional consolidation theory may be use to determine settlements of cohesive soils.

The one-dimensional consolidation test is used to determine the parameters for use in one-dimensional consolidation theory. These parameters are indicated in Table 17-7.

Table 17-7, Consolidation Parameters and Symbols

Symbol	Parameter
C_c or $C_{\varepsilon c}$	Compression Index
C_r or $C_{\varepsilon r}$	Recompression Index
C_α or $C_{\varepsilon\alpha}$	Secondary Compression Index
σ'_p or p'_c	Effective Preconsolidation Stress
c_v	Coefficient of Consolidation
m_v	Coefficient of Vertical Compression

The Coefficient of Consolidation and the Coefficient of Vertical Compression are required when determining the rate of settlement and will be discussed in the next Section. The effective preconsolidation stress is extremely important because it is used to determine if a soil is normally consolidated (NC) or overconsolidated (OC). In normally consolidated soils, the effective preconsolidation stress is equal to the existing effective overburden stress (i.e. $\sigma'_{vo} = \sigma'_p$) (see Figure 17-19). Normally consolidated soils tend to have large settlements. Overconsolidated soils tend to have an effective preconsolidation stress greater than the existing effective overburden stress (i.e. $\sigma'_{vo} < \sigma'_p$) (see Figure 17-20). Overconsolidated soils do not tend to have large settlements. One-dimensional consolidation tests shall be performed in accordance with Chapter 5. In some locations within South Carolina, under consolidated soils (i.e. $\sigma'_{vo} > \sigma'_p$) (see Figure 17-21) are known to exist. These soils are still consolidating under the weight of the soil and should be anticipated to have very large amounts of settlement.

Casagrande (1936) developed a graphical procedure for determining the preconsolidation stress. The Casagrande procedure for determining preconsolidation stress is outlined in Table 17-8. While the Casagrande procedure was applicable to both e-log p and ε-log p curves, SCDOT prefers the use of the ε-log p curve for data presentation.

Table 17-8, Determination of Preconsolidation Stress (Duncan and Buchignani – 1976)

Step	Description
1	Locate the point of sharpest curvature on the e-log p or ε-log p curve
2	From this point (a) (see Figures 17-22 or 17-23), draw a horizontal line (b) and a tangent (b) to the curve
3	Bisect the angle formed by these two lines (c)
4	Extend the virgin curve (d) backward to intersect the bisector (c)
5	The point where these lines (d and c) cross determines the preconsolidation pressure (σ'_p or p'_c)

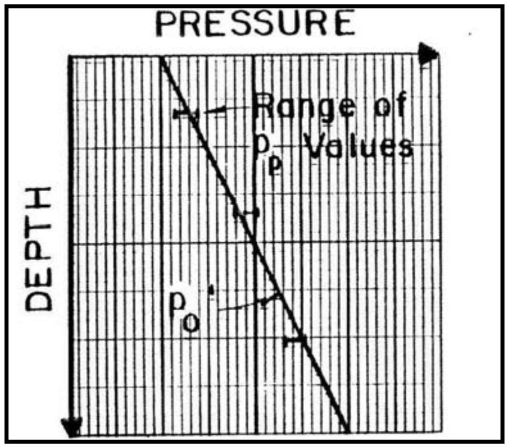


Figure 17-19, Normally Consolidated
(Duncan and Buchignani – 1976)

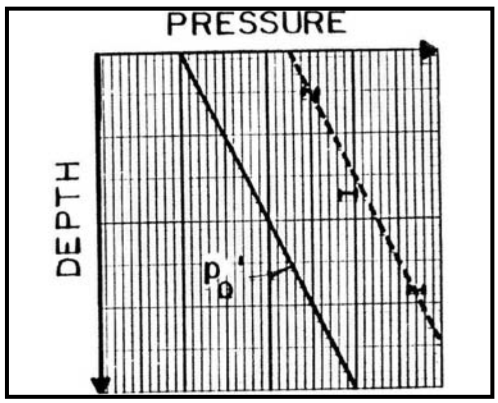


Figure 17-20, Overconsolidated
(Duncan and Buchignani – 1976)

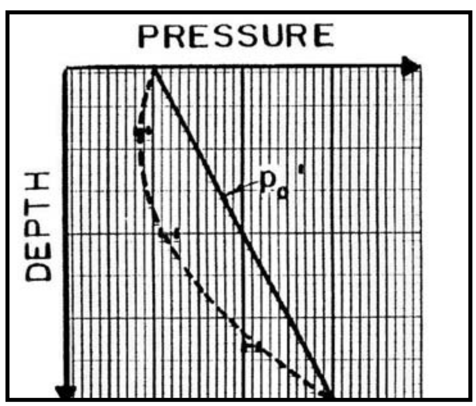


Figure 17-21, Under Consolidated
(Duncan and Buchignani – 1976)

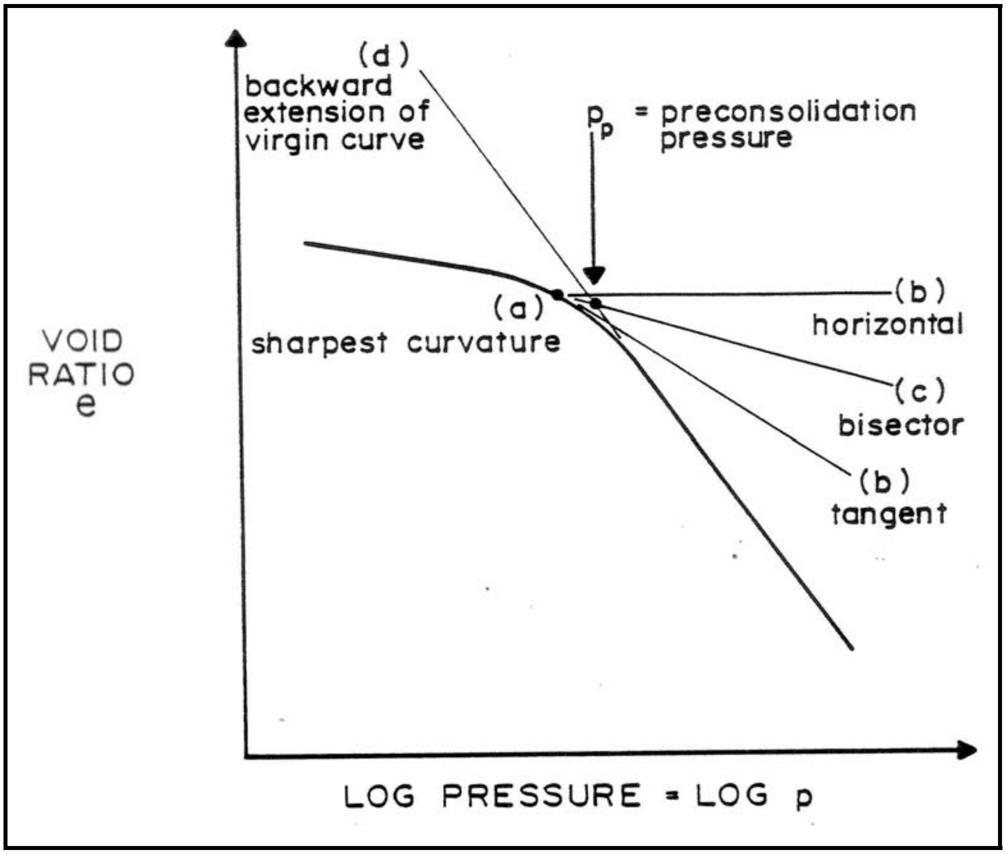


Figure 17-22, Determination of Preconsolidation Stress from e-log p (Duncan and Buchignani – 1976)

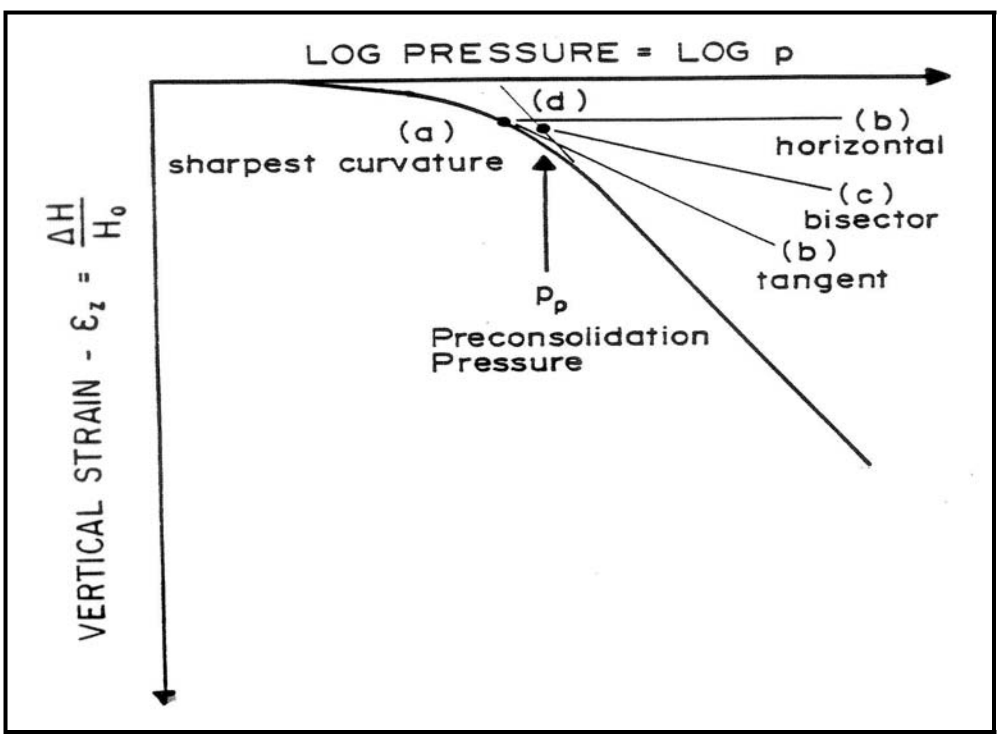


Figure 17-23, Determination of Preconsolidation Stress from ε-log p (Duncan and Buchignani – 1976)

One-dimensional consolidation tests are sensitive to sample disturbance; therefore, the results of the test must be corrected. This method is applied to test results presented as e-log p and ε-log p curves. Duncan and Buchignani (1976) provide methods for correcting both e-log p and ε-log p for both normally consolidated and overconsolidated soils. The procedures for correcting the e-log p curves (normally consolidated and overconsolidated) are presented in Table 17-9.

Table 17-9, Correction of the e-log p Curve for Disturbance (modified from Duncan and Buchignani – 1976)

Step	Description
Normally Consolidated Soil ($\sigma'_{vo} = \sigma'_p$) (Figure 17-24)	
1	Locate point A at the intersection of e_o and σ'_p (P_p)
2	Locate point B at on the virgin curve or extension where e = $0.4e_o$
3	Connect points A and B with a straight line – this is the corrected virgin curve
Overconsolidated Soil ($\sigma'_{vo} < \sigma'_p$) (Figure 17-25)	
1	Locate point A at the intersection of e_o and σ'_{vo} (P_o')
2	Draw a line from point A parallel to the rebound curve and locate point B where this line intersects σ'_p (P_p)
3	Locate point C at on the virgin curve or extension where e = $0.4e_o$
4	Connect points B and C with a straight line – this is the corrected virgin curve

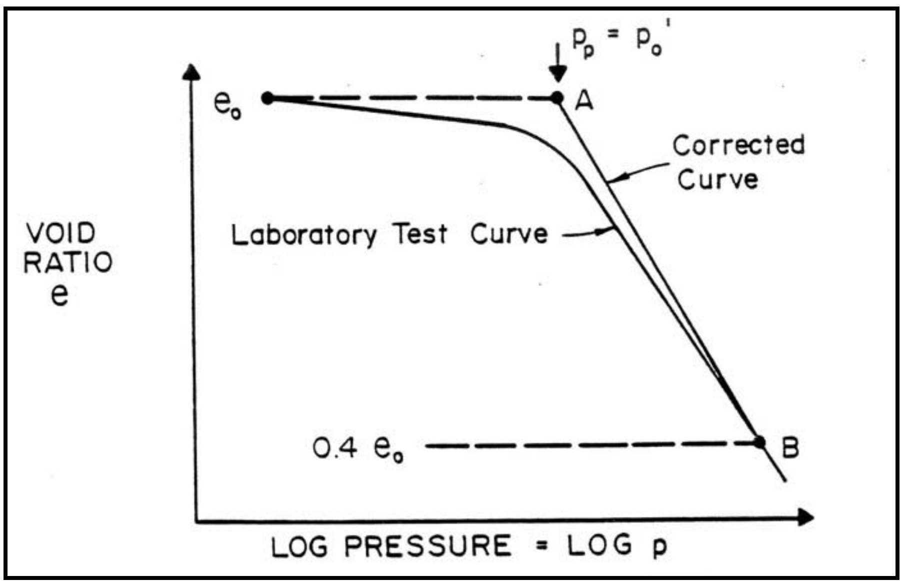


Figure 17-24, Corrected e-log p Normally Consolidated Curve (Duncan and Buchignani – 1976)

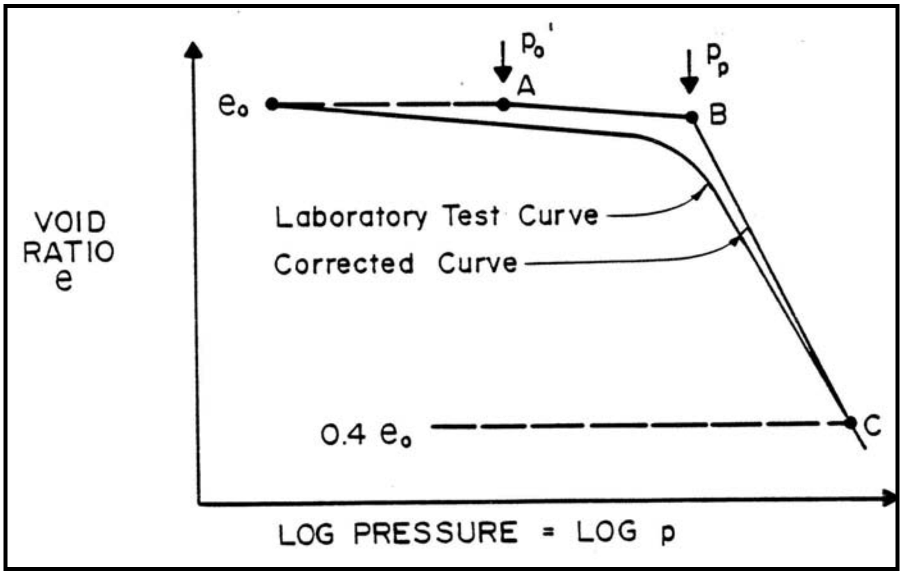


Figure 17-25, Corrected e-log p Overconsolidated Curve (Duncan and Buchignani – 1976)

The Duncan and Buchignani (1976) procedures for correcting the ε-log p curves (normally consolidated and overconsolidated) are presented in Table 17-10.

Table 17-10, Correction of the ε-log p Curve for Disturbance (modified from Duncan and Buchignani – 1976)

Step	Description
Normally Consolidated Soil ($\sigma'_{vo} = \sigma'_p$) (Figure 17-26)	
1	Locate point A at the intersection of ε = 0 and σ'_p (P_p)
2	Locate point B at on the virgin curve or extension where ε = 0.4
3	Contact points A and B with a straight line – this is the corrected virgin curve
Overconsolidated Soil ($\sigma'_{vo} < \sigma'_p$) (Figure 17-27)	
1	Locate point A at the intersection of ε = 0 and σ'_{vo} (P_o')
2	Draw a line from point A parallel to the rebound curve and locate point B where this line intersects σ'_p (P_p)
3	Locate point C at on the virgin curve or extension where ε = 0.4
4	Contact points B and C with a straight line – this is the corrected virgin curve

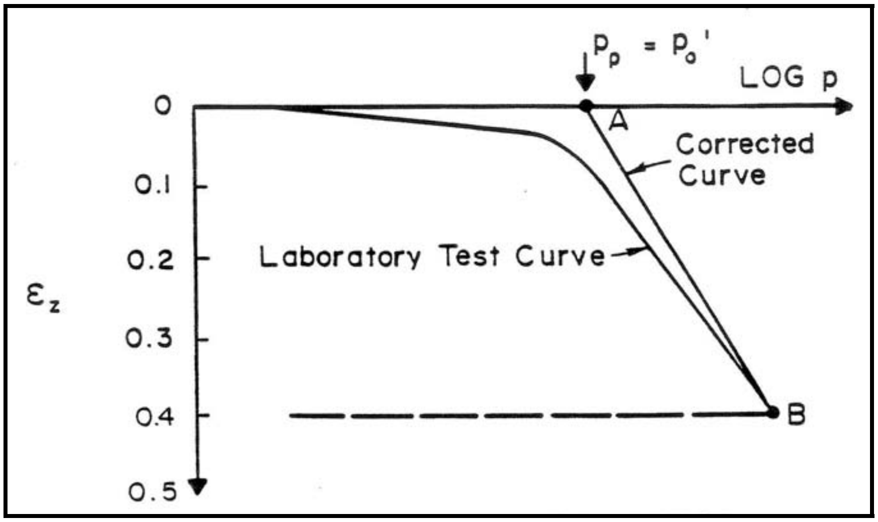


Figure 17-26, Corrected ε-log p Normally Consolidated Curve (Duncan and Buchignani – 1976)

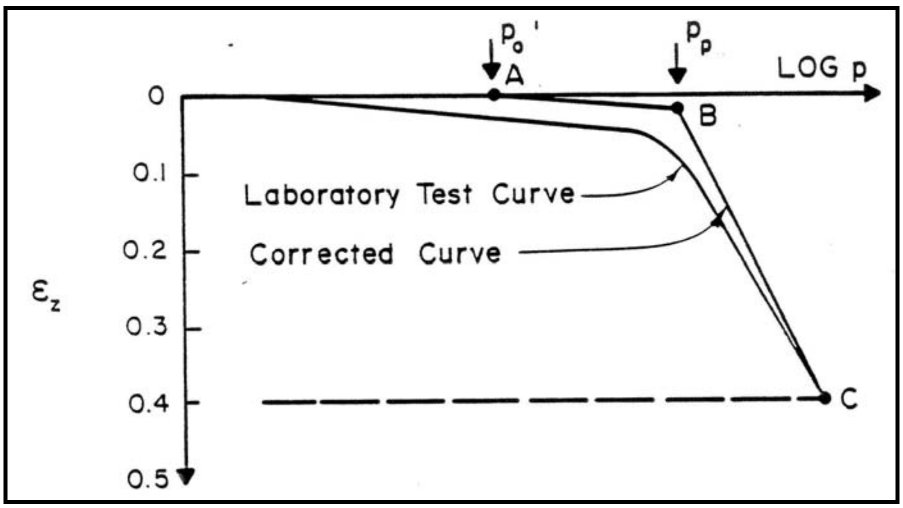


Figure 17-27, Corrected ε-log p Overconsolidated Curve (Duncan and Buchignani – 1976)

The compression (C_c or $C_{\varepsilon c}$) and recompression (C_r or $C_{\varepsilon r}$) indices are determined from the corrected curves. The compression (C_c or $C_{\varepsilon c}$) index is the slope of the virgin portion of the corrected curve, either e-log p (C_c) or ε-log p ($C_{\varepsilon c}$), over a full cycle. The recompression index is the slope of the recompression portion of the corrected curve, either e-log p (C_r) or ε-log p ($C_{\varepsilon r}$) over a full logarithmic cycle. If the slope of either portion of the curve does not extend over a full logarithmic cycle extend the line in both directions to cover a full logarithmic cycle.

The determination of the amount of settlement is dependent on whether the soil is normally consolidated, overconsolidated or a combination of both. The amount of settlement for under consolidated soils is determined the same as normally consolidated soil. In addition, the way the data is presented, either e-log p or ε-log p curves, will also determine which equation is used. Presented in Table 17-11 are the equations for determining the total primary consolidation settlement (S_c).

Table 17-11, Primary Consolidation Settlement Equations

e-log p		
$\sigma'_{vo} = \sigma'_p$	$S_c = \sum_1^i H_o \frac{C_c}{1+e_o}(\log\frac{\sigma'_f}{\sigma'_{vo}})$	Equation 17-31
$\sigma'_f < \sigma'_p$	$S_c = \sum_1^i H_o \frac{C_r}{1+e_o}(\log\frac{\sigma'_f}{\sigma'_{vo}})$	Equation 17-32
$\sigma'_{vo} < \sigma'_p < \sigma'_f$	$S_c = \sum_1^i H_o[\frac{C_c}{1+e_o}(\log\frac{\sigma'_f}{\sigma'_p}) + \frac{C_r}{1+e_o}(\log\frac{\sigma'_p}{\sigma'_{vo}})]$	Equation 17-33
ε-log p		
$\sigma'_{vo} = \sigma'_p$	$S_c = \sum_1^i H_o C_{\varepsilon c}(\log\frac{\sigma'_f}{\sigma'_{vo}})$	Equation 17-34
$\sigma'_f < \sigma'_p$	$S_c = \sum_1^i H_o C_{\varepsilon r}(\log\frac{\sigma'_f}{\sigma'_{vo}})$	Equation 17-35
$\sigma'_{vo} < \sigma'_p < \sigma'_f$	$S_c = \sum_1^i H_o[C_{\varepsilon c}(\log\frac{\sigma'_f}{\sigma'_p}) + C_{\varepsilon r}(\log\frac{\sigma'_p}{\sigma'_{vo}})]$	Equation 17-36

Where,

H_o = Thickness of i[th] layer

e_o = Initial void ratio of i[th] layer

σ'_f = Final pressure on the i[th] layer

$$\sigma'_f = \sigma'_{vo} + \Delta\sigma_v \qquad \text{Equation 17-37}$$

Where,

σ'_{vo} = initial vertical effective stress on the i[th] layer

$\Delta\sigma_v$ = change in stress on the i[th] layer

17.9.2 <u>Time for Settlement</u>

As indicated previously, consolidation settlement occurs when a load is applied to a saturated cohesive soil squeezing water out from between the soil grains. The length of time for primary consolidation settlement to occur is a function of compressibility and permeability of the soil. The Coefficient of Consolidation (c_v) is related to the permeability (k) and the Coefficient of Vertical Compression (m_v) as indicated in the following equations

$$c_v = \frac{1}{\gamma_w}\frac{k}{m_v} \qquad \text{Equation 17-38}$$

$$m_v = \frac{\Delta\varepsilon_v}{\Delta\sigma'_v}$$ Equation 17-39

$$m_v = \frac{\Delta e}{\Delta\sigma'_v\,(1+e_{av})}$$ Equation 17-40

Where,

γ_w = Unit weight of water

$\Delta\varepsilon_v$ = Change in sample height

$\Delta\sigma'$ = Change in effective stress

Δe = Change in void ratio

e_{av} = Average void ratio during consolidation

The Coefficient of Consolidation is typically provided as part of the results of consolidation testing. A c_v is provided for each load increment applied during the test. The c_v used to determine the time for primary consolidation settlement should be the one at the stress (load increment) closest to the anticipated field conditions. If c_v is not provided, it should be determined using the procedures outlined in Shallow Foundations, FHWA NHI-01-023.

The time (t) for primary consolidation settlement is determined using the equation listed below.

$$t = \frac{TH_o^2}{c_v}$$ Equation 17-41

Where,

t = time for settlement

T = Time Factor from Equation 17-42

H_o = Maximum distance pore water must flow through

c_v = Coefficient of Consolidation

The distance the pore water must flow through is affected by the permeability of the materials above and below the cohesive material. If the cohesive material is between two cohesionless materials (i.e. highly permeable materials) then the thickness of the cohesive material is cut in half. This is also known as two-way or double drainage. If the cohesive, material is bordered by an impermeable material, then the drainage path is the full thickness of the layer. This is also called one-way or single drainage. According to Das (1990), the time factor (T) is related to the degree of consolidation (U) in the following equations.

$$T = \frac{\left(\frac{\pi}{4}\right)\left(\frac{U\%}{100}\right)^2}{\left[1-\left(\frac{U\%}{100}\right)^{5.6}\right]^{0.357}} \le 2.5$$ Equation 17-42

for

$$0 \le U\% < 100\%$$ Equation 17-43

Where,

U% = Degree of Consolidation in percent

When U equals one hundred percent, T approaches infinity (∞). The limit of T indicated in Equation 17-42 results in a U of 99.3%. If the amount of settlement at a U of 99.3% exceeds the performance limits provided in Chapter 10, contact the GDS for guidance.

17.10 SECONDARY COMPRESSION SETTLEMENT

Secondary compression settlement occurs after the completion of primary consolidation settlement (i.e. U = 99.3%). This type of compression settlement occurs when the soil continues to vertically displace despite the fact that the excess pore pressures have essentially dissipated. Secondary compression typically occurs in highly plastic (PI > 21) or organic (percent organics > 30 percent) cohesive soils. Secondary compression settlement is evident on both the e-log p and the ε-log p curves (see Figure 17-28).

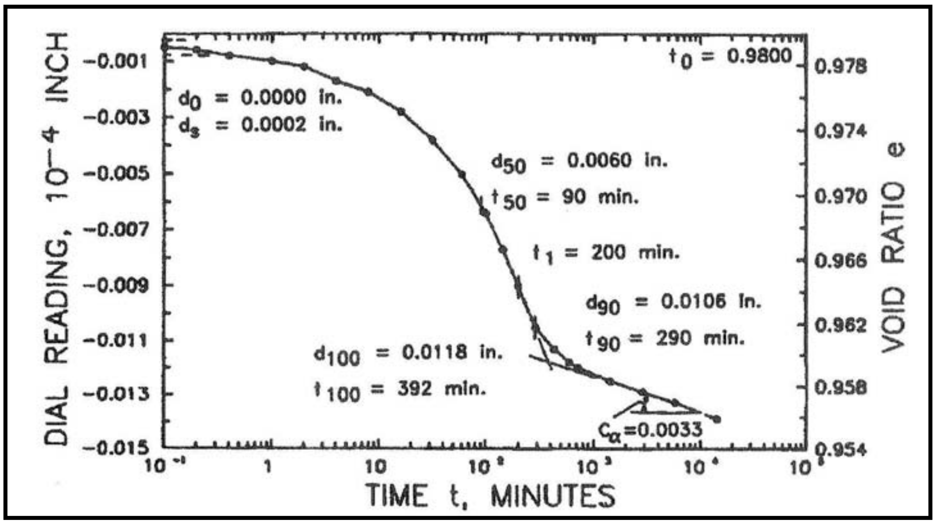


Figure 17-28, Secondary Compression (Shallow Foundations – June 2001)

The Coefficient of Secondary Compression (C_α) can be determined using the slope of the corrected curve over one full logarithmic cycle. As with primary consolidation settlement, the amount of secondary compression settlement can be determined using either the e-log p or ε-log p curves. Presented in Table 17-12 are the equations for determining secondary compression settlement.

Table 17-12, Secondary Compression Settlement Equations

e-log p	$S_s = \sum_1^i H_o \frac{C_\alpha}{1+e_o} \log\left(\frac{t_2}{t_1}\right)$	Equation 17-44
ε-log p	$S_s = \sum_1^i H_o C_{\varepsilon\alpha} \log\left(\frac{t_2}{t_1}\right)$	Equation 17-45

Where,

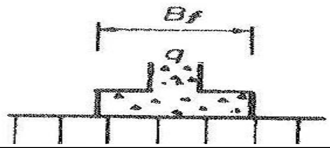
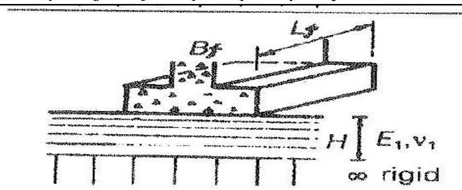
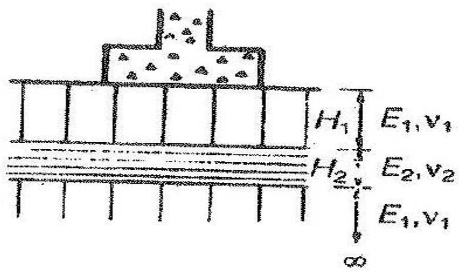
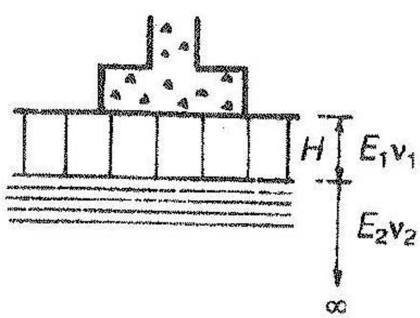
H_o = Thickness of i^{th} layer
e_o = Initial void ratio of i^{th} layer
t_1 = Time when secondary compression begins (i.e. U = 99.3%)
t_2 = Time when secondary compression is desired, usually the service life of structure

Secondary compression settlement is sometimes confused with creep. As indicated previously, secondary compression settlement occurs after the pore pressures have achieved a steady state condition and the settlement is the result of particle movement or realignment. Creep occurs after the pore pressures have achieved a steady state condition and there is no volume change. Creep is related to shear strength rather than compressibility. In many cases, it is not possible to distinguish between creep and secondary compression settlement.

17.11 SETTLEMENT IN ROCK

The settlement procedures discussed previously are for soil. Rock is normally considered incompressible; however, the potential for settlement on rock does exist. The settlement of foundations on rock can be determined using elastic theory. The settlement determinations provided in this section cover four combinations of rock (incompressible) and compressible layers (see Table 17-13).

Table 17-13, Rock Settlements on various Geological Conditions
(Shallow Foundations – June 2001)

Geological Condition	Graph of Geological Condition	Settlement Calculations
Incompressible Layer		a. Determine shape factor C_d from Table 17-14; b. Calculate S_r using equation 17-46
Compressible Layer on Incompressible Layer		a. Determine ratio of H/B_f & L_f/B_f; b. Determine shape factor C_d from Table 17-14; c. Calculate S_r using equation 17-46
Compressible Layer between Incompressible Layers		a. Determine rations $(H_1 + H_2)/B_f$ & L_f/B_f; b. Calculate weighted modulus (E) for upper two layers using equation 17-47; c. Determine shape factor C'_d, for ratio $(H_1 + H_2)/B_f$ from Table 17-15; d. Calculate S_r using equation 17-46
Incompressible Layer on Compressible Layer		a. Determine ratios H/B_f & E_1/E_2; b. Determine factor α from Table 17-16; c. Determine shape factor C_d from Table 17-14; d. Calculate $S_{r\infty}$ using equation 17-46 using elastic parameters E_2 & ν_2 for overall foundation; e. Calculate S_r by reducing calculated $S_{r\infty}$ by α (see equation 17-48)

$$S_r = \frac{C_d q B_f \left(1-\nu^2\right)}{E_m}$$ Equation 17-46

$$E = \frac{\left(E_1 H_1 + E_2 H_2\right)}{\left(H_1 + H_2\right)}$$ Equation 17-47

$$S_r = \alpha S_{r\infty}$$ Equation 17-48

Where,

S_r = Rock Settlement

$S_{r\infty}$ = Settlement of incompressible layer underlain by a compressible layer

C_d = Shape Factor from Table 17-14

C'_d = Shape Factor from Table 17-15

q_0 = Applied vertical stress at the bottom of loaded area

B_f = Foundation width

ν = Poisson's ratio

E_m =Modulus of elasticity of rock mass (see Chapter 7)

E_1 = Modulus of elasticity of incompressible layer

E_2 = Modulus of elasticity of compressible layer

α = Elastic distortion settlement correction factor

Table 17-14, Shape Factors, C_d
(Shallow Foundations – June 2001)

Shape	Center	Corner	Middle of Short Side	Middle of Long Side	Average
Circle	1.00	0.64	0.64	0.64	0.85
Circle (rigid)	0.79	0.79	0.79	0.79	0.79
Square	1.12	0.56	0.76	0.76	0.95
Square (rigid)	0.99	0.99	0.99	0.99	0.99
Rectangle: Length/Width					
1.5	1.36	0.67	0.89	0.97	1.15
2	1.52	0.76	0.98	1.12	1.30
3	1.78	0.88	1.11	1.35	1.52
5	2.10	1.05	1.27	1.68	1.83
10	2.53	1.26	1.49	2.212	2.25
100	4.00	2.00	2.20	3.60	3.70
1000	5.47	2.75	2.94	5.03	5.15
10000	6.90	3.50	3.70	6.50	6.60

Table 17-15, Shape Factors, C'_d
(Shallow Foundations – June 2001)

H/B_f	Circle Dia. (B_f)	Rectangle Shape Foundation (L_f/B_f)						
		1	1.5	2	3	5	10	∞
0.10	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
0.25	0.24	0.24	0.23	0.23	0.23	0.23	0.23	0.23
0.50	0.48	.048	0.47	0.47	0.47	0.47	0.47	0.47
1.00	0.70	0.75	0.81	0.83	0.83	0.83	0.83	0.83
1.50	0.80	0.86	0.97	1.03	1.07	1.08	1.08	1.08
2.50	0.88	0.97	1.12	1.22	1.33	1.39	1.40	1.40
3.50	0.91	1.01	1.19	1.31	1.45	1.56	1.59	1.60
5.00	0.94	1.05	1.24	1.38	1.55	1.72	1.82	1.83
∞	1.00	1.12	1.36	1.52	1.78	2.10	2.53	∞

Table 17-16, Elastic Distortion Settlement Correction Factor, α
(Shallow Foundations – June 2001)

H/B_f	E_1/E_2				
	1	2	5	10	100
0.00	1.000	1.000	1.000	1.000	1.000
0.10	1.000	0.972	0.943	0.923	0.760
0.25	1.000	0.885	0.779	0.699	0.431
0.50	1.000	0.747	0.566	0.463	0.228
1.00	1.000	0.627	0.399	0.287	0.121
2.5	1.000	0.550	0.274	0.175	0.058
5.0	1.000	0.525	0.238	0.136	0.036
∞	1.000	0.500	0.200	0.100	0.010

17.12 LATERAL SQUEEZE

Lateral squeeze is a phenomenon that occurs when a soft cohesive soil deforms and displaces when subjected to embankment loadings and is primarily a concern at the end bents where the deep foundations may be installed through thick layers of soft cohesive soils. In addition, if the thickness of the soft cohesive soils is finite and is less then the width of the embankment (b_e) the potential for lateral squeeze is present (Figure 17-14). This phenomenon can cause rotation and horizontal displacement of the end bent and can induce excessive loadings in the deep foundations. The following equation is used to determine if the potential for lateral squeeze exists at a site.

$$\gamma_f H_f > 3\tau \qquad \text{Equation 17-49}$$

Where,

γ_f = Total unit weight of fill material

H_f = Height of fill

τ = Undrained shear strength (see Chapter 7)

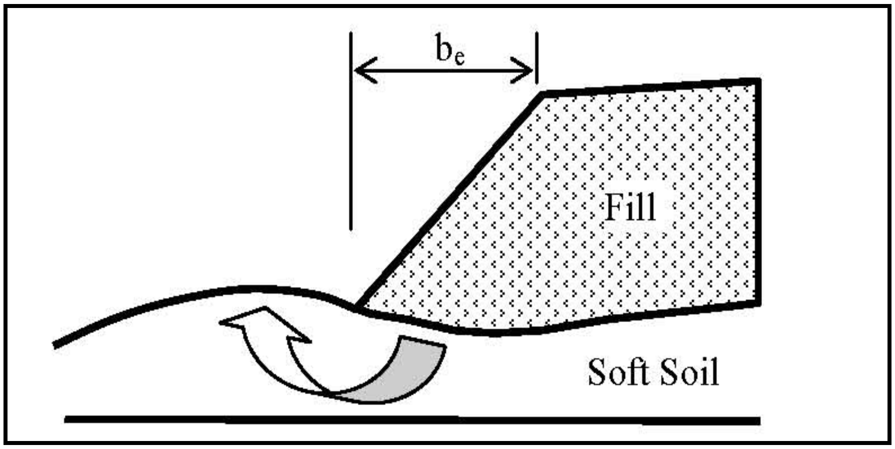


Figure 17-29, Schematic of Lateral Squeeze (Soils and Foundations – December 2006)

If the load applied by the soil (fill height times fill unit weight) exceeds three times the undrained shear strength the potential for lateral squeeze is present; therefore, lateral movements of the soil may occur. These lateral movements may be estimated using the following equation.

$$\Delta_L = 0.25S_t \qquad \text{Equation 17-50}$$

Where,

Δ_L = Horizontal displacement

S_t = Total settlement of fill

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Chapter 18

EARTH RETAINING STRUCTURES

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

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CHAPTER 18

EARTH RETAINING STRUCTURES

18.1 INTRODUCTION

Earth retaining structures (ERSs) are used to retain earth materials while maintaining a grade change between the front and rear of the wall (see Figure 18-1). ERSs transmit the loads (Q_1, Q_2, and p_1) to a combination of the base and the restraining element (p_2 and deadman) to maintain stability. Typically, ERSs are expensive when compared to embankments; therefore, the need for an ERS should be carefully considered in preliminary design. An effort should be made to keep the retained soil height to a minimum. ERSs are used to support cut and fill slopes where space is not available for construction of flatter more stable slopes (see Chapter 17). Bridge abutments and foundation walls are designed as ERSs since these structures are used to support earth fills.

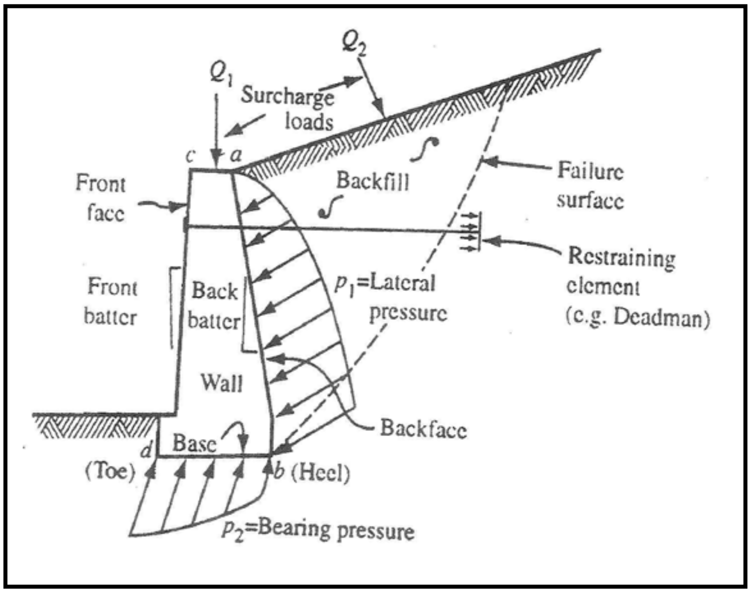

Figure 18-1, Retaining Wall Schematic (Earth Retaining Structures – June 2008)

According to <u>Earth Retaining Structures</u> (FHWA-NHI-07-071) dated June 2008, ERSs are typically used in highway construction for the following applications:

- New or widened highways in developed areas
- New or widened highways at mountains or steep slopes
- Grade separations
- Bridge abutments, wing walls and approach embankments
- Culvert walls
- Tunnel portals and approaches
- Flood walls, bulkheads and waterfront structures
- Cofferdams for construction of bridge foundations
- Stabilization of new or existing slopes and protection against rockfalls
- Groundwater cut-off barriers for excavations or depressed roadways

18.2 EARTH RETAINING STRUCTURE CLASSIFICATION

There are four criteria for classifying an ERS:

- Load support mechanism (externally or internally stabilized walls)
- Construction concept (fill or cut)
- System rigidity (rigid or flexible)
- Service life (permanent or temporary)

All ERSs are classified using all four of the criteria listed above; however, the service life is not normally used since most ERSs are designed as permanent. For example, a soldier pile and lagging wall is classified as an externally stabilized flexible cut wall, while a soil nail wall is an internally stabilized flexible cut wall. The design of temporary ERSs is discussed at the end of this Chapter. Therefore, the intermediate Sections are concerned with the design of permanent ERSs. Figure 18-2 provides a partial representation of the classification of permanent ERSs; this Figure is partial in that it does not include all of the possible types of walls available.

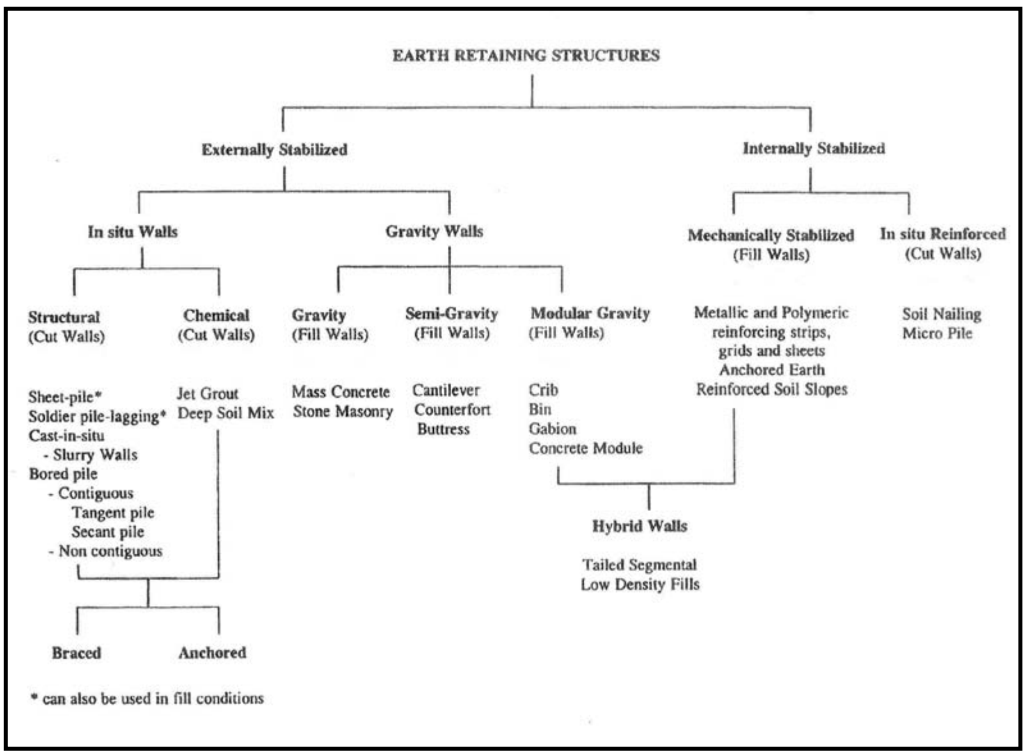

Figure 18-2, ERS Classification Chart
(Modified from Earth Retaining Structures – June 2008)

18.2.1 <u>Load Support Mechanism Classification</u>

The load support mechanism classification is based on whether the ERS is stabilized externally or internally. Externally stabilized ERSs use an external structure against which the stabilizing forces are mobilized. Internally stabilized ERSs use reinforcements that are installed within the soil mass and extend beyond the potential failure surface to mobilize the stabilizing forces.

18.2.2 Construction Concept Classification

ERSs are also classified based on the construction method used. The construction methods consist of fill or cut. Fill construction refers to an ERS that is constructed from the base to the top (i.e. bottom-up construction). Conversely, cut construction refers to an ERS that is constructed from the top to the base (i.e. top-down construction). It is very important to realize the cut or fill designations refer to how the ERS is constructed, not the nature of the earthwork. For example, a prefabricated bin wall could be placed in front of a "cut" slope, but the wall would be classified as a "fill" wall since the construction is from the bottom-up.

18.2.3 System Rigidity Classification

The rigidity of the ERS is fundamental to understanding the development of the earth pressures that develop behind and act on the ERS. A rigid ERS moves as a unit (i.e. rigid body rotation and/or translation) and does not experience bending deformations. A flexible ERS undergoes not only rigid body rotation and/or translation, but also experiences bending deformations. In flexible ERSs, the deformations allow for the redistribution of the lateral (earth) pressures from the more flexible portion of the wall to the more rigid portion of the wall. Most gravity type ERSs would be considered an example of a rigid wall. Almost all of the remaining ERS systems would be considered flexible.

18.2.4 Service Life Classification

The focus of this Chapter is on permanent ERS construction. According to Chapter 10, all geotechnical structures including ERS shall have a design life of 100 years. Temporary ERSs shall have a service life less than 5 years. Temporary ERSs that are to remain in service more than 5 years shall be designed as a permanent ERS. A more detailed explanation of temporary ERSs is provided at the end of this Chapter.

18.3 LRFD ERS DESIGN

The design of ERSs is comprised of two basic components, external and internal. External design handles stability, sliding, and bearing; while internal design handles pullout failure of soil anchors or reinforcement and structural failure of the ERS. The external stability of an ERS is checked using the procedures outlined in Chapter 17. For ERSs supported by shallow foundations, sliding and bearing are checked using Chapter 15, while those ERSs supported by deep foundations are checked using Chapter 16. All loads that affect the external stability of an ERS shall be developed using Chapter 8, as well as the procedures outlined in AASHTO LRFD Bridge Design Specifications (latest edition), Article 11.5 – Limit States and Resistance Factors. Where there is conflict, this Manual takes precedence over AASHTO. According to Earth Retaining Structures, FHWA – June 2008; "In general, use minimum load factors if permanent loads increase stability and use maximum load factors if permanent loads reduce stability." The resistance factors shall be developed using Chapter 9 for both Strength and Service limit states. Chapter 9 divides ERSs into three types of walls; Rigid Gravity, Flexible Gravity and Cantilever ERSs. Chapter 9 provides examples of the different types of common walls that fit within each group. Cantilever ERSs are a sub-class of Gravity ERSs per Figure 18-2. In accordance with Chapter 8, the Service limit state is the boundary condition for performance of the structure under service load conditions. The Service limit state is evaluated for the movements induced by the Service load combinations (see Chapter 8). The movements induced by the Service loads are compared to the performance objectives established in Chapter 10. At the end of construction, the ERS shall have a front batter that either meets the performance objectives indicated in Chapter 10 or is vertically plumb. Once all of the external designs are checked, then, the internal design is checked. ERSs comprised of MSE walls or Reinforced Soil Slopes (RSSs) use the internal resistance factors as presented in Chapter 9.

All ERS designs must meet the requirements of the basic LRFD equation

$$\mathbf{Q} = \sum \gamma_{\mathbf{i}} \mathbf{Q}_{\mathbf{i}} \leq \varphi_{\mathbf{n}} \mathbf{R}_{\mathbf{n}} = \mathbf{R}_{\mathbf{r}} \qquad \text{Equation 18-1}$$

Where,

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

18.4 ERS SELECTION PHILOSOPHY

The selection of the type of ERS is based on numerous factors. It is possible for more than one ERS type to be applicable to a given site. Figure 18-3 provides a flow chart for determining the most appropriate type of wall for a specific location. Further, Tables 18-1 and 18-2 provide the most common cut and fill walls (see discussion above on ERS classification). The ERSs listed in Tables 18-1 and 18-2 contain walls that are typically used by SCDOT and walls that would be allowed. Written permission to use walls other than those indicated in these tables shall be obtained from the GDS and the PCS/GDS prior to designing the wall.

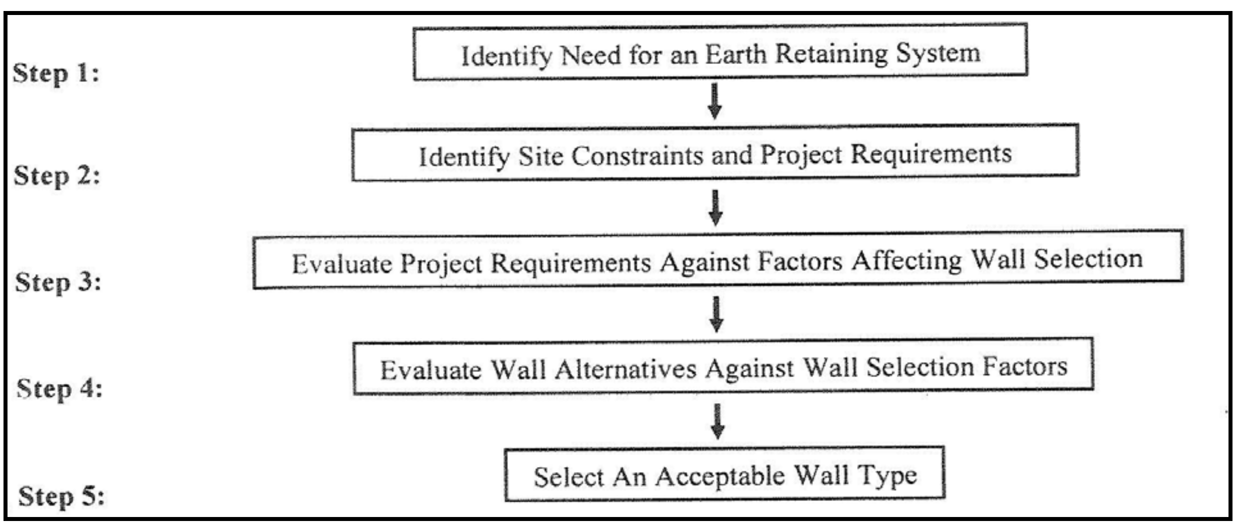

Figure 18-3, Wall Selection Flow Chart
(Earth Retaining Structures – June 2008)

Table 18-1, Cut Wall Evaluation Factors
(Earth Retaining Structures – June 2008)

	Wall Type	Application[1]	Height Range[2]	Required ROW[4]	Lateral Movements	Advantages	Disadvantages
CANTILEVERED	Sheet-pile	P/T	<16 ft	None	Large	- Rapid construction - Readily Available	- Difficult to construct in hard ground or penetrate obstructions
	Soldier Pile/Lagging	P/T	<16 ft	None	Medium	- Rapid construction - Soldier piles can be drilled or driven	- Difficult to maintain vertical tolerances in hard ground - Potential for ground loss at excavated face
	Anchored	P/T	15 - 70 ft	0.6H + abl[3]	Small-medium	- Can resist large horizontal pressures - Adaptable to varying site conditions	- Requires skilled labor and specialized equipment - Anchors may require permanent easements
IN-SITU REINFORCED	Soil-nailed	P/T	10 – 70 ft	1.0H	Small-medium	- Rapid construction - Adaptable to irregular wall alignment	- Nails may require permanent easements - Difficult to construct and design below water table
	Micropile	P	N/A	Varies	N/A	- Does not require excavation	- Requires specialty contractor

[1]P/T – Permanent and Temporary
[2]Height range based on cost effectiveness
[3]abl – Anchor Bond Length
[4]ROW requirements expressed as the distance (as a fraction of wall height, H) behind the wall face where anchorage components are installed

Table 18-2, Fill Wall Evaluation Factors
(Earth Retaining Structures – June 2008)

	Wall Type	Application[1]	Height Range[2]	Required ROW[3]	Differential Settlement Tolerance	Advantages	Disadvantages
RIGID GRAVITY	Gravity[4]	P	3 – 10 ft	0.7H	1/500	- Durable - Requires smaller quantity of select backfill as compared to MSE walls - Can meet aesthetic requirements	- Deep foundation support may be necessary - Relatively long construction time
	Cantilever[4]	P	6 – 30 ft	0.7H	1/500	- Durable - Requires smaller quantity of select backfill as compared to MSE walls - Can meet aesthetic requirements	- Deep foundation support may be necessary - Relatively long construction time
	Counterfort[4]	P	30 – 60 ft	0.7H	1/500	- Durable - Requires smaller quantity of select backfill as compared to MSE walls - Can meet aesthetic requirements	- Deep foundation support may be necessary - Relatively long construction time
FLEXIBLE GRAVITY	Gabion	P/T	6 – 30 ft	0.7H	1/50	- Does not require skilled labor or equipment	- Need adequate source of stone - Construction of wall requires significant labor
	MSE Wall – precast facing	P/T	10 - 100 ft	1.0H	1/100	- Does not require skilled labor or equipment - Flexibility in choice of facing	- Requires use of select backfill - Subject to corrosion in aggressive environments (metallic reinforcement)
	MSE Wall – modular block facing	P/T	6 – 60 ft	1.0H	1/200	- Does not require skilled labor or equipment - Flexibility in choice of facing - Blocks are easily handled	- Requires use of select backfill - Subject to corrosion in aggressive environments (metallic reinforcement) - Positive reinforcement connection to blocks is difficult to achieve
	MSE Wall – geotextile / geogrid / welded wire facing	P/T	6 – 50 ft	1.0H	1/60	- Does not require skilled labor or equipment - Flexibility in choice of facing	- Facing may be aesthetically pleasing -Geosynthetic reinforcement is subject to degradation in some environments
	MSE Wall – vegetated soil face	P/T	10 – 100 ft	1.0H	1/60	- Does not require skilled labor or equipment - Flexibility in choice of facing - Vegetation provides ultraviolet light protection to geosynthetic reinforcement	- Facing may be aesthetically pleasing -Geosynthetic reinforcement is subject to degradation in some environments - Vegetated soil face requires significant maintenance

[1]P/T – Permanent and Temporary
[2]Height range based on cost effectiveness
[3]ROW requirements expressed as the distance (as a fraction of wall height, H) behind the wall face where anchorage components are installed
[4]These walls are all constructed of cast-in-place concrete and/or standard brick and mortar

18.4.1 Necessity for ERS

As indicated in Figure 18-3, the first step in selecting an ERS type is to determine if a wall is needed. According to the SCDOT *Highway Design Manual* (HDM) (2003 with latest revisions), the need for ERSs is determined jointly by the Program Manager and the Road Design Team. Typically, ERSs are required in areas where additional Right-of-Way (ROW) cannot be obtained or there are other factors (i.e. other roads, major utilities, etc.) that limit the development of stable slopes. The need for ERSs can often be determined during the DFR (see Chapter 1).

18.4.2 Site Constraints and Project Requirements

Once the need for an ERS is identified, then specific site constraints and project requirements need to be identified. Listed below are some items that will affect ERS selection. This list is not all inclusive.

1. Site accessibility and space restrictions
    a. Limited ROW
    b. Limited headroom
    c. On-site material storage areas
    d. Access for specialized construction equipment
    e. Traffic Disruption restrictions
2. Utility locations, both above and underground
3. Nearby structures
4. Aesthetic requirements
5. Environmental concerns
    a. Construction noise
    b. Construction vibration
    c. Construction dust
    d. On-site stockpiling, transport and disposal of excavated materials
    e. Discharge of large volumes of water
    f. Encroachment on existing waterways
6. Exposed wall face height

The relative importance of each of these items should be assessed by the Project Team for the specific project under consideration. This assessment should identify those items that should be given priority in the selection process.

18.4.3 Factors Affecting ERS Selection

Step 3 from Figure 18-3 establishes the process for evaluating project requirements against fairly common factors that affect the selection of an ERS. Twelve factors have been identified and indicated in Table 18-3. The factors are listed in no particular order. Additional factors may be considered based on the requirements of the Project Team. Each factor is evaluated based on its relevancy and importance to the project requirements and site constraints. Each factor is assigned a number rating from one, the least relevant, to three, the most relevant. The rating is termed the weighted rating (WR). Table 18-4 depicts an example of the selection factors and WR for each factor.

Table 18-3, ERS Selection Factors
(Modified from Earth Retaining Structures – June 2008)

1	Ground type	7	Environmental concerns
2	Groundwater	8	Durability and maintenance
3	Construction considerations	9	Tradition
4	Speed of construction	10	Contracting practices
5	ROW	11	Cost
6	Aesthetics	12	Displacements (lateral and vertical)

Table 18-4, Weighted ERS Selection Factors
(Modified from Earth Retaining Structures – June 2008)

	Ground Type	Groundwater	Construction Considerations	Speed of Construction	ROW	Aesthetics	Environmental Concerns	Durability and Maintenance	Tradition	Contracting Practice	Cost	Displace.
WR[1]												

[1]Wieghted rating of the importance of each ERS selection factor based on project requirements and site constraints.
Each factor should be rated between 1, least importance factor, and 3, most important factor.

18.4.4 Evaluate ERS Alternates

The fourth step in selecting an ERS type consists of reviewing specific ERS types versus the Weighted ERS Selection Factors presented in the previous section. A logical first step in this process is the elimination of ERS types that would be inappropriate for the specific project site. This elimination process should focus on project constraints such as ERS geometry and performance; however, the project constraints related to costs should not be included as a reason to eliminate an ERS type. In addition, the factors affecting cut (top-down construction) or fill (bottom-up construction) ERS selection should also be evaluated.

The selection issues discussed in this Section apply to permanent ERSs, selection issues for temporary cut walls are discussed later in this Chapter. Typically, permanent cut walls are designed with greater corrosion protection or with higher strength materials. In addition, these types of ERSs have permanent facing elements that consist of either cast-in-place concrete or precast concrete panels. Cut ERSs are typically either cut or drilled into the existing geomaterials at a site and require specialty contractors. If ground anchors are not required, then little or no ROW is required. However, if anchors or soil nails are used, then either additional ROW or permanent easements will be required. The taller a cut ERS becomes, the higher the unit cost of the ERS becomes. Depending on the geotechnical conditions, for ERS heights ranging from 15 to 30 feet or greater, either anchors or soil nails will be required. Cut ERSs typically used by SCDOT are provided in Table 18-1.

Fill ERSs are constructed from the bottom-up and are typically used for permanent construction. However, temporary MSE walls can also be constructed using flexible facing elements. Fill ERSs typically require more ROW than cut ERSs. Typically, the soil used for fill ERSs is comprised of granular, nonplastic, free draining geomaterials. The requirements for high quality fill materials typically increase the cost of fill ERSs. Fill ERSs typically used by SCDOT are provided in Table 18-2.

Those ERS systems not eliminated earlier in this step should be evaluated using the ERS Selection Factors (see Table 18-3). An Importance Selection Factor (ISF) rating of one to four is applied to each ERS Selection Factor for each ERS type. An ISF rating of four means the factor is most suitable for the ERS type under evaluation, while an ISF rating of one means the factor is least suitable. Any cost associated with a selection factor should be considered when developing the rating. A brief description of each selection factor is provided in the following sections.

18.4.4.1 Ground Type

According to Earth Retaining Structures, FHWA - June 2008, “An *ERS* is influenced by the earth it is designed to retain, and the one on which it rests.” For ERSs that are internally supported (MSE walls and soil nail walls), the quality of the retained soil in which the reinforcement is placed is of great influence. For MSE walls, the pull-out force of the reinforcement is developed by the friction along the soil-reinforcement interface and any passive resistance that develops along transverse members of the reinforcement, if any are present. Typically, MSE walls require high quality granular fill materials with relatively high friction angles. Plastic fine-grained soils (i.e. cohesive) are not used in MSE wall design or construction. For soil nail walls used to support excavations, the possible saturation and creep associated with plastic fine-grained soils can have a negative impact on the performance of the structure. For externally supported ERSs (gravity, semi-gravity and modular gravity), the influence of the retained soil is less important. However, for soils that undergo large vertical and horizontal displacements, a flexible ERS (i.e. gabion) should be used in lieu of a more rigid ERS. A rigid ERS will attempt to resist the movements, thereby placing more stress on the structural members.

18.4.4.2 Groundwater

The groundwater table behind ERSs should be lowered for the following reasons:

1. To reduce the hydrostatic pressures on the structure
2. To reduce the potential for corrosion of metal reinforcing in the retained soil
3. To reduce the potential for corrosion of metal reinforcing in facing elements
4. To prevent saturation of the soil
5. To limit displacements that can be caused by saturated soils
6. To reduce the potential for soil migration through or from the ERS

Typically, fill ERSs are constructed with free-draining backfill, while the ERS face contains numerous weep holes or other means for water to be removed from behind the structure. Drainage media is also installed in cut walls. An SCDOT ERS shall never be designed to retain water. If the necessity for water retention is mandated on a project, the PCS/GDS should be contacted for instructions and guidance.

18.4.4.3 Construction Considerations

Construction considerations that need to be accounted for in the selection of an ERS are material availability, site accessibility, equipment availability and labor considerations. The availability of construction materials can affect selection of an ERS. For example, the use of a gabion wall in Charleston would not be practical since all stone for the gabion would have to be hauled in, while on the other hand a gabion wall in Cherokee could be successfully used. Limited site accessibility could limit the type of ERS that could be constructed. Another construction consideration is the requirement for specialized equipment and is the equipment locally or at least regionally available. The final construction consideration is the labor force to be used to build the wall (i.e. does the labor force require specialized training?).

18.4.4.4 Speed of Construction

Another factor to be considered is the speed at which the ERS can be constructed. The more rapidly an ERS can be constructed, the more rapidly the project can be completed.

18.4.4.5 Right-of-Way

The amount or need for additional Right-of-Way (ROW) should be considered when selecting an ERS wall type. The question that needs to be asked is whether the ERS is being used to support the transportation facility or to support an adjacent owner. ERSs supporting the transportation facility should require limited to no additional ROW, while ERSs supporting an adjacent owner may require either additional ROW or an easement to install ground anchors, etc.

18.4.4.6 Aesthetics

Depending on the location of the ERS, the aesthetics of the wall can be of great importance in final selection. Typically, the aesthetics of wall is more important in populated areas than in non, or limited, populated areas. In more environmentally sensitive areas, the ERS may need to blend in with the surrounding environment. This need for blending should be accounted for in ERS selection.

18.4.4.7 Environmental Concerns

ERSs can both cause, as well as alleviate, environmental concerns. ERSs cause environmental concerns if contaminated soil must be removed prior to or during the construction of the structure. In addition, noise and vibration from certain ERS wall installations can have a negative impact on the environment around the project. In addition, the fascia of some ERSs may allow for the bouncing or echoing of traffic noise; therefore, in cases where this may become a concern, an alternate fascia material may need to be selected. ERSs may alleviate environmental concerns by allowing for smaller footprints in environmentally sensitive areas; therefore, eliminating the need for environmental permits.

18.4.4.8 Durability and Maintenance

Depending on the environmental conditions (corrosiveness) of materials the ERS is founded on or is constructed of, certain ERS types may not be satisfactory. The ERS must be durable for the life of the structure (100 years) or must have definitive maintenance procedures that will need to be identified. These maintenance procedures should also clearly indicate the time periods that maintenance should be performed.

18.4.4.9 Tradition

Tradition (i.e. what is normally done) can impact what type of ERS is selected. Traditionally, SCDOT uses the following wall types:

1. MSE
2. RSS
3. Cantilever (concrete or standard brick/block)
4. Soil Nail
5. Sheetpile (cantilever or anchored)
6. Soldier pile and lagging (cantilever or anchored)
7. Gabion
8. Gravity

18.4.4.10 Contracting Practices

The use of sole source or patented ERSs should be avoided at all times. If sole source or patented ERSs cannot be avoided, a written justification is required. The written justification shall be maintained in the project file and shall include the endorsement (approval) of the Regional Production Engineer.

18.4.4.11 Cost

The total cost of the ERS should include the structure (structural elements and backfill materials, if any), ROW (acquisition or easement), excavation and disposal of unsuitable or contaminated materials, mitigation costs of environmental impacts (such as additional noise) and the time value of construction delays. Credits for eliminating environmental permits or speeding up construction should as be factored into the decision.

18.4.4.12 Displacements

The amount of displacement (horizontal and vertical) that an ERS may be required to handle also affects the selection process. Some walls are more flexible than others. An idea of the amount of displacement that an ERS is anticipated to endure should also be known prior to making the final ERS selection.

18.4.5 Selection of Acceptable ERS Type

The final step in selecting an ERS is to determine the most acceptable type. This determination is made based on the ISF rating and the weighted rating each of the above selection factors is given for each ERS type. A score is arrived at by multiplying the ISF and the WR for each selection factor and summing all of the results. The ERS with the highest score is most acceptable type and should be developed in design. Other highly scored walls may be included in the Contract Documents as acceptable alternatives. Table 18-5 provides an example of this process.

Table 18-5, Wall Selection Matrix
(Modified from Earth Retaining Structures – June 2008)

	Ground	Groundwater	Construction Considerations	Speed of Construction	ROW	Aesthetics	Environmental Concerns	Durability and Maintenance	Tradition	Contracting Practice	Cost	Displacement	Total Score
ISF[1]	3	2	2	3	1	1	2	2	3	1	3	3	
MSE Wall – Precast Facing	4	1	3	3	1	4	4	3	4	2	3	2	77
Gabion Wall	3	1	3	1	4	1	4	4	1	3	1	4	62
Cast-in-place Concrete Gravity Wall	4	1	1	1	1	4	2	4	3	4	1	2	58

[1]Importance selection factor (ISF) for each ERS are based on project requirements and site constraints. Each factor should rated be between 1, least importance factor, and 3, most important factor.

18.5 EARTH PRESSURE THEORY

Earth pressures act on the rear face of an ERS and are caused by the weight of the soil (backfill or retained fill), seismic loads and various surcharge loads. The ERS is designed to resist these loads, as well as, any water (pore) pressures that may build up on the rear of the wall. There are three different lateral pressures used in the design of ERSs (see Table 18-6).

Table 18-6, Earth Pressure Definitions

Earth Pressure	Symbol	Definition
Active	K_a	The lateral pressure that is developed when the wall moves away from the backfill resulting in a decrease in pressure on wall relative to the at-rest pressure
At-Rest	K_o	The lateral pressure that exists in level ground for condition of no lateral deformation
Passive	K_p	The lateral pressure that is developed when the wall moves toward the backfill resulting in an increase in pressure on wall relative to the at-rest pressure

The general horizontal earth pressure is expressed by the following equation.

$$\sigma_{\boldsymbol{h}} = \boldsymbol{K}\sigma_{\boldsymbol{v}}$$ Equation 18-2

Where,

σ_h = Horizontal earth pressure at a specific depth on an ERS
K = Earth pressure coefficient
σ_v = Vertical earth pressure (overburden stress) at a specific depth on an ERS

The active and passive earth pressure coefficients are a function of the soil shear strength, backfill geometry, the geometry of the rear face of the ERS and friction and cohesion that develop along the rear face as the wall moves relative to the retained backfill. The active earth pressure condition is developed by a relatively small movement of the ERS away from the retained backfill, while the movements required to develop the passive earth pressure condition are on the order to approximately ten times larger than the movements required to develop active conditions (see Figure 18-4).

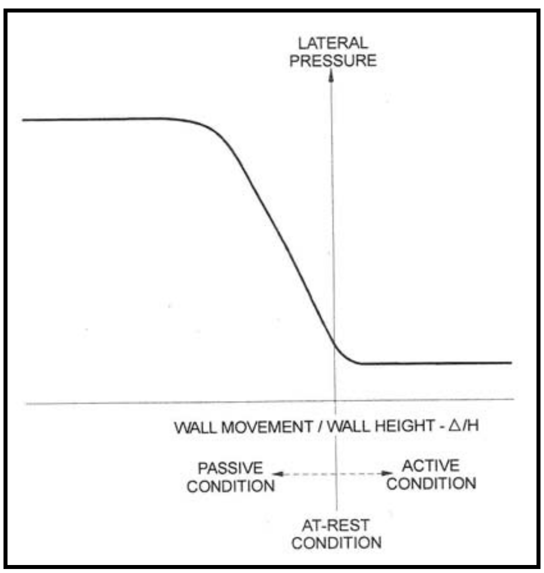

Figure 18-4, Relative Magnitude of Displace. Required to Develop Earth Pressures (Earth Retaining Structures – June 2008)

18.5.1 Active Earth Pressure

As indicated previously, the active earth pressure condition exists when the ERS is free to rotate away from the retained backfill. There are two earth pressure theories available for determining the earth pressure coefficients; Rankine and Coulomb earth pressure theories. Rankine earth pressure makes several assumptions concerning the wall and the backfill. The first assumption is that the retained soil has a horizontal surface, secondly, that the failure surface is a plane and finally that the wall is smooth (i.e. no friction). Rankine earth pressure theory is the preferred method for developing the active earth pressure coefficient; however, under the appropriate conditions Coulomb earth pressure theory may be used. The use of Rankine theory will cause a slight over estimation of K_a, therefore, increasing the pressure on the wall resulting in a more conservative design. The equations for developing the active earth pressure coefficient for cohesionless and cohesive soils, respectively, are indicated below:

$$K_a = \tan^2\left(45 - \frac{\varphi'}{2}\right) \quad \text{Equation 18-3}$$

$$K_a = \tan^2\left(45 - \frac{\varphi'}{2}\right) - \frac{2c'}{\sigma'_v}\tan^2\left(45 - \frac{\varphi'}{2}\right) \quad \text{Equation 18-4}$$

Where,

φ' = Effective friction angle
c' = Effective cohesion
σ'_v = Effective overburden pressure at bottom of wall

ERSs used on SCDOT projects shall be designed to prevent the buildup of pore water pressures behind the wall. The effective active pressure on an ERS shall be determined using the following equations for cohesionless and cohesive soils, respectively:

$$p_a' = K_a(\gamma z - u) \quad \text{Equation 18-5}$$

$$p_a' = K_a(\gamma z - u) - 2c'\sqrt{K_a} \quad \text{Equation 18-6}$$

Where,

γ = Total unit weight of soil
z = Depth of interest
u = Static pore water pressure
K_a = Active earth pressure coefficient
c' = Effective cohesion

The active pressure may also be expressed in terms of total stress by adding the pore pressure to the effective active pressure. This condition will only exist immediately after the completion of construction. If the soil has $c=s_u$ and $\varphi=0$, then total active pressure equation becomes:

$$p_a = \gamma z - 2s_u \qquad \text{Equation 18-7}$$

18.5.2 At-Rest Earth Pressure

In the at-rest earth pressure (K_o) condition, the top of the ERS is not allowed to deflect or rotate; therefore, requiring the wall to support the full pressure of the soil behind the wall. The at-rest earth pressure coefficient is related to the OCR (Chapter 7) of the soil. The following equation is used to determine the at-rest earth pressure coefficient:

$$K_o = (1 - \sin\varphi')(OCR)^{\Omega} \qquad \text{Equation 18-8}$$

$$\Omega = \sin\varphi' \qquad \text{Equation 18-9}$$

While all soils can be overconsolidated, the ability to accurately determine the OCR for cohesionless soil is not cost effective; therefore, the OCR for all cohesionless materials shall be taken as 1.0. Therefore for cohesionless materials, Equation 18-8 may be rewritten as:

$$K_o = 1 - \sin\varphi' \qquad \text{Equation 18-10}$$

Flexible walls are not typically designed to withstand the at-rest earth pressure condition. In this case, the effective at-rest earth pressure is determined using the following equation for both cohesionless and cohesive soils:

$$p_o' = K_o(\gamma z - u) \qquad \text{Equation 18-11}$$

18.5.3 Passive Earth Pressure

The development of passive earth pressure requires the ERS to move into or toward the soil. As with the active earth pressure, Rankine earth pressure is the preferred method to be used to develop passive earth pressure coefficient. Coulomb earth pressure theory may be used if the appropriate conditions exist at a site; however, the designer is required to understand the limitations on the use of Coulomb earth pressure theory as applied to passive earth pressures. The use of Rankine theory will cause an under estimation of K_p, therefore resulting in a more conservative design. The equations for developing the passive earth pressure coefficient for cohesionless and cohesive soils, respectively, are indicated below:

$$K_p = \tan^2\left(45 + \frac{\varphi'}{2}\right) \qquad \text{Equation 18-12}$$

$$K_p = \tan^2\left(45 + \frac{\varphi'}{2}\right) + \frac{2c'}{\sigma'_v}\tan^2\left(45 + \frac{\varphi'}{2}\right) \qquad \text{Equation 18-13}$$

Where,

φ' = Effective friction angle

c' = Effective cohesion

σ'_v = Effective overburden pressure at bottom of wall

ERSs used on SCDOT projects shall be designed to prevent the buildup of pore water pressures behind the wall. The effective passive pressure on an ERS shall be determined using the following equations for cohesionless and cohesive soils, respectively:

$$p_p' = K_p(\gamma z - u) \quad \text{Equation 18-14}$$

$$p_p' = K_p(\gamma z - u) + 2c'\sqrt{K_p} \quad \text{Equation 18-15}$$

Where,

- γ = Total unit weight of soil
- z = Depth of interest
- u = Static pore water pressure
- K_p = Passive earth pressure coefficient
- c' = Effective cohesion

The passive pressure may also be expressed in terms of total stress by adding the pore pressure to the effective passive pressure. This condition will only exist immediately after the completion of construction. If the soil has $c=s_u$ and $\varphi=0$, then total passive pressure equation becomes:

$$p_p = \gamma z + 2s_u \quad \text{Equation 18-16}$$

18.6 GRAVITY RETAINING WALLS

Gravity ERSs are externally stabilized fill walls and consist of the wall types provided in Table 18-7. Gravity wall types can be subdivided into three categories; gravity, semi-gravity and modular gravity. The limited details of each wall type are discussed in the following Sections. The design of gravity retaining walls is also discussed.

Table 18-7, Gravity Wall Types

Gravity	Semi-Gravity	Modular Gravity
Mass Concrete	Cantilever	Gabion
Stone	Counterfort	Crib
Masonry	Buttress	Bin

18.6.1 Gravity Retaining Walls

Gravity walls are typically trapezoidal in shape; although for shorter walls, the walls are more rectangular (SCDOT Standard masonry walls) (see Figure 18-5). Gravity walls are constructed of either mass concrete with little or no reinforcement or masonry or stone walls. These types of walls tend to behave rigidly and depend on the weight (mass) of concrete to resist overturning and sliding.

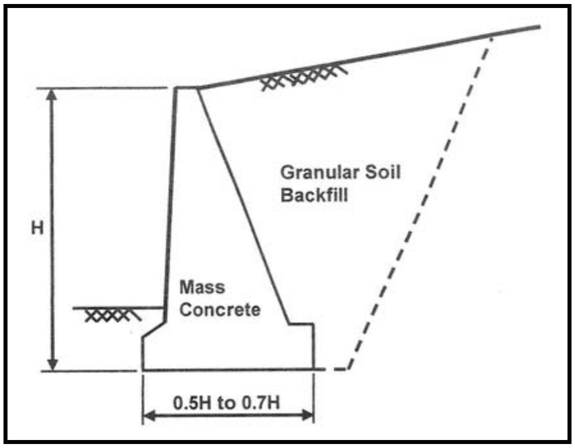

Figure 18-5, Gravity Retaining Wall
(Earth Retaining Structures – June 2008)

18.6.2 Semi-Gravity Retaining Walls

Semi-gravity walls are comprised of cantilevered, counterfort or buttress walls (see Figure 18-6). Semi-gravity walls are constructed of reinforced concrete, with the reinforcing in the stem designed to withstand the moments induced by the retained soil. Typically, cantilevered walls are limited to heights less than 30 feet. The counterforts (buttress within the retained soil mass) or buttresses (buttress on exposed face of the wall) are used when the moments are too large requiring a thicker stem and more reinforcing. Typically, these types of walls are used when the cantilevered wall height exceeds 30 feet.

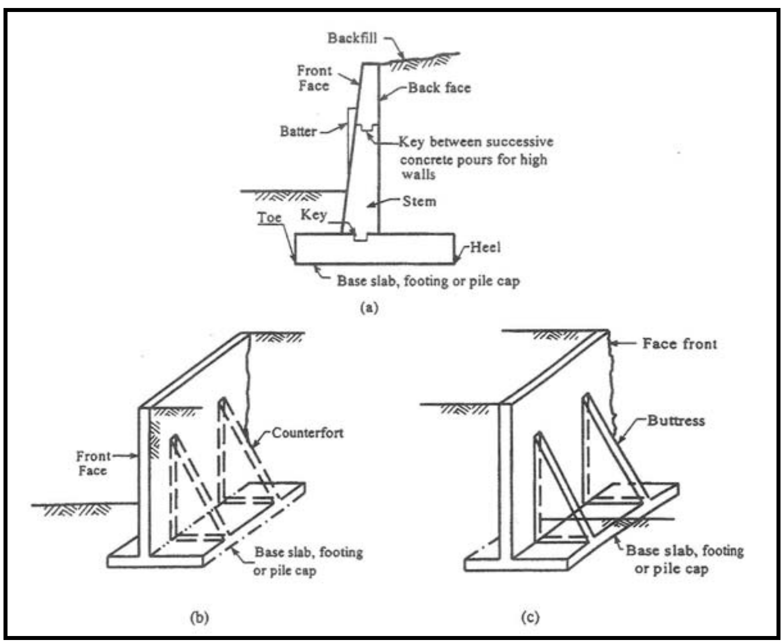

Figure 18-6, Semi-Gravity Retaining Wall
(Earth Retaining Structures – June 2008)
(a) Cantilever; (b) Counterfort; (c) Buttress

18.6.3 Modular Gravity Walls

Modular gravity walls are comprised of gabion, crib or bin walls (see Figure 18-7). Gabion walls are rock filled wire baskets. Gabion walls are used in locations where rock is plentiful. These types of walls are labor intensive to construct. Gabion walls are often used in applications that will experience cycles of inundation from streams. Currently SCDOT does not use crib or bin walls. The use of crib or bin walls must be approved in writing by the GDS and the PCS/GDS prior to commencing design.

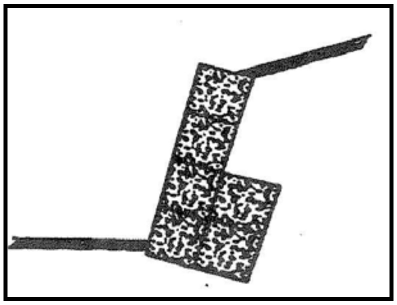

Figure 18-7, Gabion Retaining Wall
(Earth Retaining Structures – June 2008)

18.6.4 Gravity Wall Design

The design of gravity ERSs includes the overall (global) stability, bearing and deformation, sliding and overturning. The overall (global) stability and deformation analyses are performed using the procedures presented in Chapter 17. The bearing, sliding and overturning analyses are performed using the procedures discussed in Chapter 15, if shallow foundations are used. If deep foundations are required, then the procedures presented in Chapter 16 should be used. Table 18-8 provides the design steps for gravity walls. For additional details on the design of gravity walls refer to Earth Retaining Structures, FHWA - June 2008. The loads placed on gravity retaining walls should be developed in accordance with Section 11 – Abutments, Piers and Walls of the latest version of the AASHTO LRFD Bridge Design Specifications and Chapter 8 of this Manual. Resistance Factors and Performance Limits shall be developed in accordance with Chapters 9 and 10 of this Manual.

Table 18-8, Gravity Wall Design Steps
(Earth Retaining Structures – June 2008)

Step	Action
1	Establish project requirements including all geometry, external loading conditions (transient and/or permanent, seismic, etc.), performance criteria and construction constraints.
2	Evaluate site subsurface conditions and relevant properties of in-situ soil and rock parameters and wall backfill parameters.
3	Evaluate soil and rock parameters for design and establish resistance factors.
4	Select initial base dimension of wall for Strength limit state (external stability) evaluation.
5	Select lateral earth pressure distribution. Evaluate water, surcharge, compaction and seismic pressures.
6	Evaluate factored loads for all appropriate loading groups and limit states.
7	Evaluate bearing resistance (Chapter 15).
8	Check eccentricity (Chapter 15).
9	Check sliding (Chapter 15).
10	Check overall stability at the Service limit state and revise wall design if necessary (Chapter 17).
11	Estimate maximum lateral wall movement, tilt, and wall settlement at the Service limit state. Revise design if necessary.
12	Design wall drainage systems.

18.7 IN-SITU STRUCTURAL WALLS

In-situ structural walls have structural elements (i.e. sheetpile or soldier pile and lagging) installed to provide resistance of the applied lateral loads (see Figure 18-8). These types of walls are externally stabilized cut (top-down construction) walls. In-situ structural walls may develop resistance to the applied lateral loads through cantilever action, anchors or internal bracing (see Figure 18-9). In typical SCDOT applications, the use of exterior bracing is not normally used and will therefore not be discussed. Two different design methods are required for in-situ structural walls depending if the wall is cantilevered or supported by anchors. Typically, cantilevered in-situ structural walls can have exposed heights of up to 15 feet. Cantilevered in-situ structural walls taller than this will require anchors to resist the bending moments induced by the soil on the structural elements. The anchors may be either deadman or tendon type, depending on the method of construction, the amount of ROW available, etc. Table 18-9 provides the design steps for a cantilevered in-situ structural wall. Anchored in-situ structural walls are designed using the steps provided in Table 18-10. For additional details on the design of in-situ structural walls refer to Earth Retaining Structures, FHWA - dated June 2008. The loads placed on in-situ structural retaining walls should be developed in accordance with Section 11 – Abutments, Piers and Walls of the latest version of the AASHTO LRFD Bridge Design Specifications and Chapter 8 of this Manual. Resistance Factors and Performance Limits shall be developed in accordance with Chapters 9 and 10 of this Manual.

Table 18-9, Cantilevered In-Situ Structural Wall Design Steps (Earth Retaining Structures – June 2008)

Step	Action
1	Establish project requirements including all geometry, external loading conditions (transient and/or permanent, seismic, etc.), performance criteria and construction constraints.
2	Evaluate site subsurface conditions and profile, water profile, and relevant properties of in-situ soil and rock parameters.
3	Evaluate soil and rock parameters for design and establish resistance factors.
4	Select lateral earth pressure distribution. Evaluate water, surcharge, compaction and seismic pressures.
5	Evaluate factored total lateral pressure diagram for all appropriate limit states.
6	Evaluate embedment depth of vertical wall element and factored bending moment in the wall.
7	Check flexural resistance of vertical wall elements. Check combined flexural and axial resistance (if necessary).
8	Select temporary lagging (for soldier pile and lagging wall).
9	Design permanent facing (if required).
10	Estimate maximum lateral wall movements and ground surface settlement at the Service limit state. Revise design if necessary.

Table 18-10, Anchored In-Situ Structural Wall Design Steps (Earth Retaining Structures – June 2008)

Step	Action
1	Establish project requirements including all geometry, external loading conditions (transient and/or permanent, seismic, etc.), performance criteria and construction constraints.
2	Evaluate site subsurface conditions and relevant properties of in-situ soil and rock parameters.
3	Evaluate soil and rock parameters for design and establish resistance factors and select level of corrosion project for the anchor.
4	Select lateral earth pressure distribution acting on back of wall for the final wall height. Evaluate water, surcharge, and seismic pressures.
5	Evaluate factored total loads for all appropriate limit states.
6	Calculate horizontal ground anchor loads and subgrade reaction force. Resolve each horizontal anchor load into a vertical force component and a force along the anchor. Evaluate horizontal spacing of anchors based on wall type and calculate individual factored anchor loads.
7	Evaluate required anchor inclination based on right-of-way limitations, location of appropriate anchoring strata, and location of underground structures.
8	Select tendon type and check tensile resistance.
9	Evaluate anchor bond length.
10	Evaluate factored bending moments and flexural resistance of wall.
11	Evaluate bearing resistance of wall below excavation subgrade. Revise wall section if necessary.

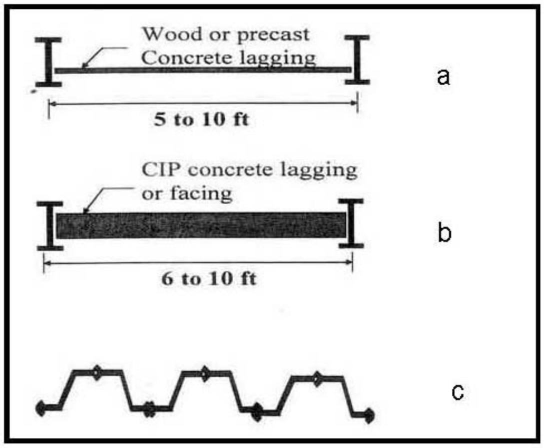

Figure 18-8, In-Situ Structural Walls
(Modified from Earth Retaining Structures – June 2008)
a and b Soldier pile and lagging; c Sheetpile

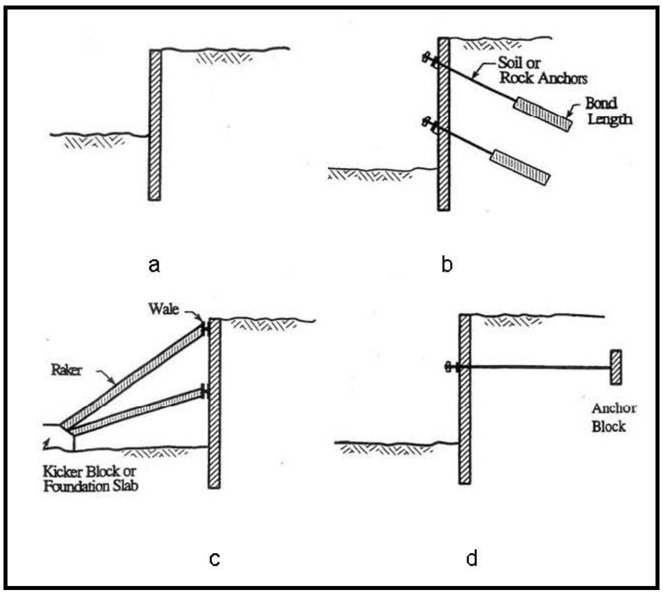

Figure 18-9, Wall Support Systems
(Modified from Earth Retaining Structures – June 2008)
a Cantilever; b Anchored; c Braced; d Deadman Anchored

18.8 MECHANICALLY STABILIZED EARTH WALLS

Mechanically Stabilized Earth (MSE) Walls are internally stabilized fill walls that are constructed using alternating layers of compacted soil and reinforcement (i.e. geogrids, metallic strips or metallic grids) (see Figure 18-10). As indicated in Table 18-2, there are four MSE Wall face alternatives that may be used when necessary. MSE Wall with precast panel facing shall be used at bridge end bent locations. However, other face options may be used with written permission of the GDS and the PCS/GDS. Table 18-11 provides the design steps that are used in the design of MSE Walls. Appendix C provides a detailed design procedure. The loads placed on in-situ structural retaining walls should be developed in accordance with Section 11 – Abutments, Piers and Walls of the latest version of the AASHTO LRFD Bridge Design Specifications and Chapter 8 of this Manual. Resistance Factors and Performance Limits shall be developed in accordance with Chapters 9 and 10 of this Manual. The external stability of the MSE Wall is the responsibility of the geotechnical engineer-of-record. The internal stability of the MSE Wall is the responsibility of either the structural designer or the MSE Wall supplier.

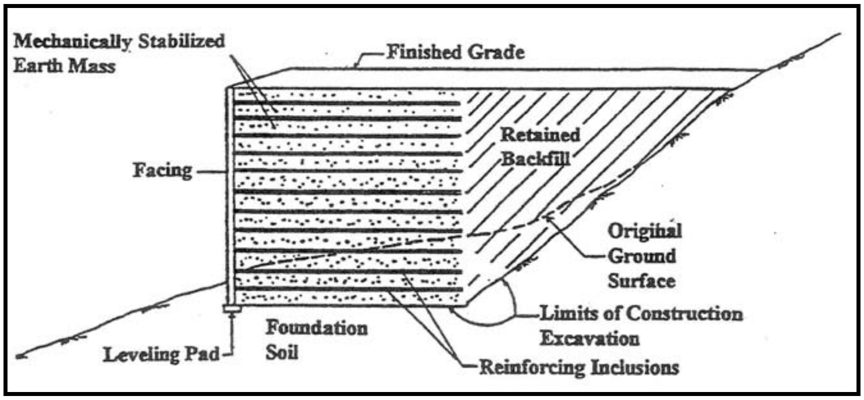

Figure 18-10, MSE Wall
(Earth Retaining Structures – June 2008)

Table 18-11, MSE Wall Design Steps
(Earth Retaining Structures – June 2008)

Step	Action
1	Establish project requirements including all geometry, external loading conditions (transient and/or permanent, seismic, etc.), performance criteria and construction constraints.
2	Evaluate existing topography, site subsurface conditions, in-situ soil/rock parameters and wall backfill parameters.
3	Based on initial wall geometry, estimate wall embedment depth and length of reinforcement.
4	Estimate unfactored loads.
5	Calculate factored loads for all appropriate limit states (external stability).
6	Check eccentricity (Appendix C).
7	Check sliding resistance (Appendix C).
8	Check bearing resistance (Appendix C).
9	Estimate critical failure surface based on reinforcement type (i.e. extensible or inextensible) for internal stability design at all appropriate Strength limit states.
10	Calculate factored horizontal stress at each reinforcement level.
11	Calculate maximum factored tensile stress in each reinforcement.
12	Check reinforcement pullout resistance.
13	Calculate nominal long-term reinforcement design strength and check reinforcement tensile resistance.
14	Check overall stability at the Service limit state (Chapter 17). Revise design if necessary.
15	Estimate vertical and lateral wall movements at the Service limit state. Revise design if necessary.
16	Design wall drainage systems.

18.9 IN-SITU REINFORCED WALLS

In-situ reinforced walls are internally stabilized cut walls that involve the insertion of reinforcing elements into the in-situ soils to create a composite ERS (see Figure 18-11).

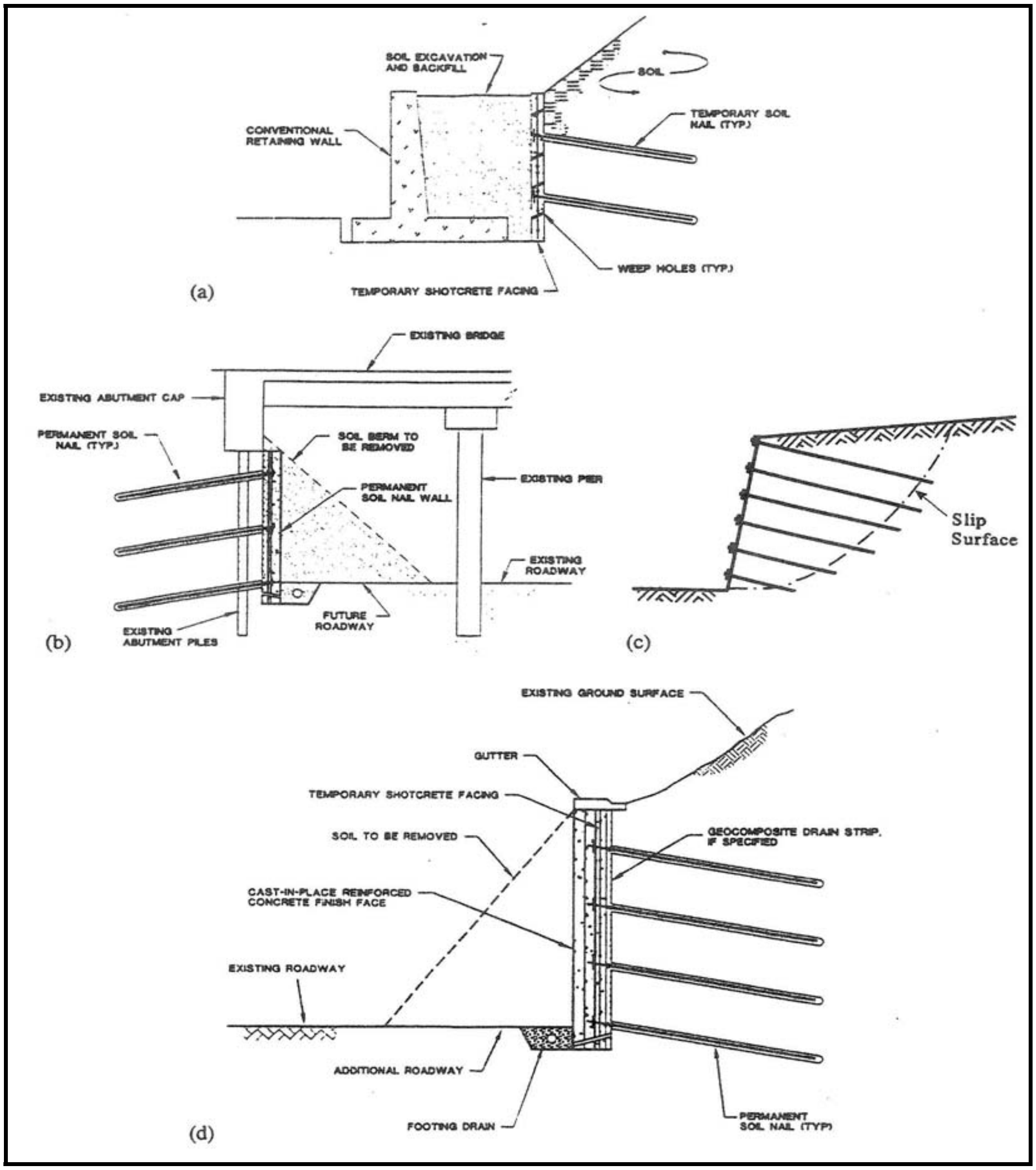

Figure 18-11, In-Situ Reinforced (Soil Nail) Walls
(Earth Retaining Structures – June 2008)
a Temporary shoring; b Roadway widening under existing bridge;
c Slope stabilization; d Roadway cut

The design steps for a soil nail wall are provided in Table 18-12. For detailed requirements of design, please refer to Earth Retaining Structures, FHWA - June 2008. An alternate detailed design source is Soil Nail Walls, FHWA - March 2003. The loads placed on in-situ structural retaining walls should be developed in accordance with Section 11 – Abutments, Piers and Walls of the latest version of the AASHTO LRFD Bridge Design Specifications and Chapter 8 of this Manual. Performance Limits and Resistance Factors shall be developed in accordance with Chapters 10 and 9 of this Manual. The external stability of the soil nail wall is the responsibility of the geotechnical engineer-of-record. The internal stability of the soil nail wall is the responsibility of either the structural designer or the soil nail wall contractor.

Table 18-12, Soil Nail Wall Design Steps
(Earth Retaining Structures – June 2008)

Step	Action
1	Establish project requirements including all geometry, external loading conditions (transient and/or permanent, seismic, etc.), performance criteria, aesthetic requirements, and construction constraints.
2	Evaluate site subsurface conditions and relevant properties of in-situ soil and rock.
3	Develop initial soil nail wall design criteria.
4	Perform preliminary design using simplified design chart solutions.
5	Evaluate external stability including global stability (Chapter 17), sliding and bearing capacity (Chapter 15).
6	Evaluate internal stability including nail pullout resistance and tensile resistance.
7	Perform facing design including: a) evaluation of nail head load; b) selection of temporary and permanent facing materials and thicknesses; c) evaluation of facing flexural resistance; d) evaluation of facing punching shear resistance; and, e) evaluation of facing stud tensile resistance.
8	Estimate maximum lateral wall movements.
9	Design wall subsurface and surface drainage systems

18.10 HYBRID WALLS

Hybrid walls are composed of two or more different types of walls or slopes (see Figure 18- 12). These kinds of walls allow a reduction in the ROW required for the construction of a project. The use of hybrid walls will require special attention from the design engineer. The various components of the hybrid wall may require different deformations to develop adequate resistance to the external loads. These differences can lead to incompatible deformations at the face of wall. The continuity of the drainage system must be maintained in both components of the hybrid wall. Finally, while the performance and design information for each component is known, the performance of the hybrid wall system is typically not known.

The combining of cut and fill walls should be performed with extreme care, since most cut walls require small strains to develop resistance, while most fill walls require larger strains to develop the same resistance. If the walls move (displace) different amounts to develop the required resistances, the face of the wall may display unaesthetic differential movements, even if the wall is structurally sound. The fact that the face shows displacement can cause the general public to consider the wall failing. In addition, the higher strains required to develop the resistance of one portion of the wall can induce higher loads in other portion of the wall causing failure of the wall.

In most cases, the hybrid wall consists of a stacked system (see Figure 18-12) with one wall or slope on top of another. The overall stability of the entire system must be checked in accordance with Chapter 17. Then, each individual wall component should be checked for stability. The lower wall should include the weight of the upper wall as a surcharge load. The design of the upper wall should include the movements (vertical and lateral) of the lower wall in design (see Chapter 17). The design engineer should have a clear understanding of how each different wall component will perform prior to selecting the use of a hybrid wall.

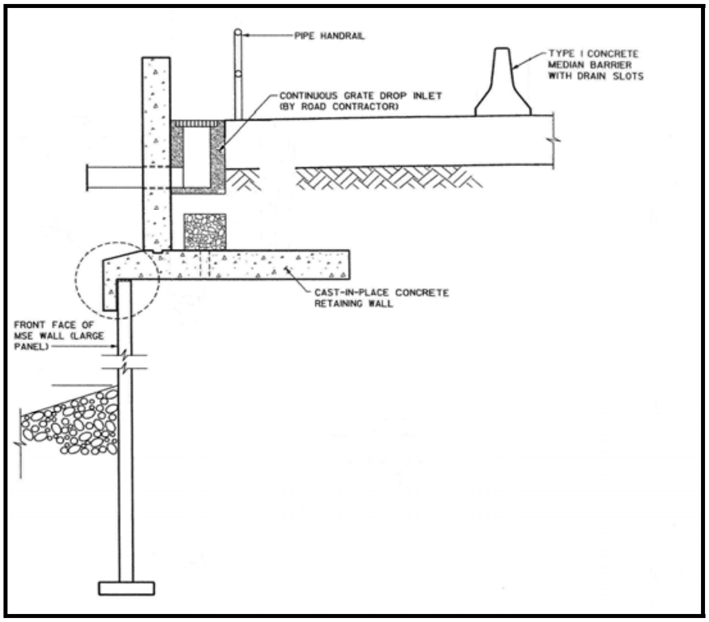

Figure 18-12, Hybrid Wall – Cantilever Concrete over MSE Wall

18.11 REINFORCED SOIL SLOPES

Steepened Reinforced Soil Slopes (RSS) are a transitional geometry between conventional (unreinforced) soil slopes and ERSs. Typically, RSSs have slopes ranging from 2H:1V to 1H:1V. Slopes flatter than 2H:1V are typically unreinforced, while structures with slopes greater than 1H:1V are considered to be ERSs. RSSs consist of reinforcement arranged in horizontal planes in the reinforced mass to resist the outward movement of this mass. The reinforcement allows the reinforced mass to act more rigid than in an unreinforced soil slope. Facing treatments can range from vegetated to flexible armor systems that are applied to prevent unraveling and sloughing of the face (see Figure 18-13). Appendix D contains detailed design methodologies for RSSs. Table 18-13 provides the design steps that are used in the design of RSS.

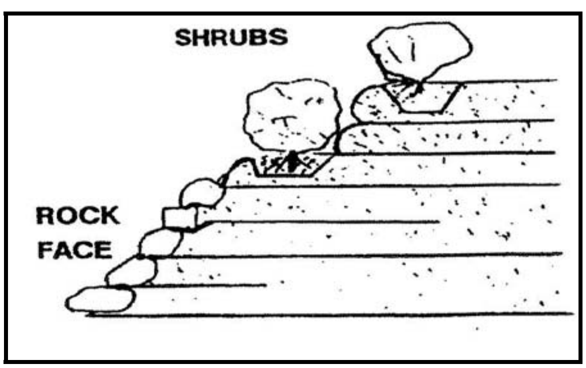

Figure 18-13, Reinforced Soil Slope (Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction – March 2001)

Table 18-13, RSS Design Steps
(modified from Mechanically Stabilized Earth Walls and
Reinforced Soil Slopes Design and Construction – March 2001

Step	Action
1	Establish project requirements including all geometry, external loading conditions (transient and/or permanent, seismic, etc.), performance criteria and construction constraints.
2	Evaluate existing topography, site subsurface conditions, and in-situ soil/rock parameters.
3	Determine properties of available fill materials.
4	Evaluate design parameters for the reinforcement.
5	Check unreinforced stability.
6	Design reinforcement to provide stable slope.
7	Determine type of reinforcement.
8	Check external stability.
9	Evaluate requirements for subsurface and surface water control.

The overall design of RSSs is similar to unreinforced slopes (see Chapter 17). However, there are three possible modes of slope failure (see Figure 18-14):

I. Internal – failure plane passes through reinforced soil mass
II. External – failure plane passes behind and underneath reinforced soil mass
III. Compound – failure plane passes behind and through reinforced soil mass

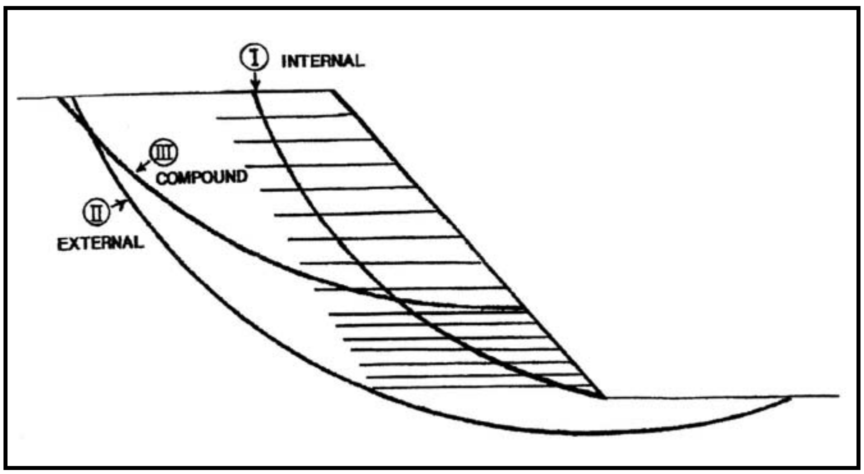

Figure 18-14, Reinforced Soil Slope Failure Modes
(Mechanically Stabilized Earth Walls and
Reinforced Soil Slopes Design and Construction – March 2001)

18.12 TEMPORARY WALLS

Temporary shoring walls are used to support a temporary excavation that is required to allow construction to proceed. Temporary shoring walls have a service of less than 5 years. Any shoring wall with a service life of greater than 5 years shall be designed as a permanent ERS.

Another major distinction between permanent and temporary ERSs is an increase in the resistance factor allowed in design. Temporary walls may be subdivided into two classes “support of excavation“ (SOE) and “critical.” SOE walls typically support just the excavation while the critical temporary walls support critical structures (i.e. existing roadway and traffic, bridge end bent fill, etc.). The resistance factors and performance limits established (see Chapters 9 and 10) are for critical temporary walls. The PCS/GDS should be contacted for the resistance factors and performance limits for SOE temporary walls. The design of temporary walls uses the same methodologies as the permanent walls.

18.13 REFERENCES

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Chapter 19

GROUND IMPROVEMENT

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

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CHAPTER 19

GROUND IMPROVEMENT

19.1 INTRODUCTION

According to Ground Improvement Methods, Volume I, FHWA NHI-06-019, August 2006, "One of the major functions of geotechnical engineering is to design, implement and evaluate ground improvement schemes for infrastructure projects. During the last 25 years significant new technologies and methods have been developed and implemented to assist the geotechnical specialist in providing cost-effective solutions for construction on marginal or difficult sites." The ground improvement methods discussed in this Chapter are based on the contents of Ground Improvement Methods, Volumes I and II but should not be the complete discussion of ground improvement methods. For simplicity Ground Improvement Methods, Volumes I and II, FHWA NHI-06-019 and FHWA NHI-06-020, August 2006 will be referenced as Ground Improvement Methods. The engineer is should consultant each volume as required for more details concerning a specific ground improvement method. In keeping with the geotechnical philosophy described in Chapter 7, it is incumbent on the geotechnical engineer/professional to be aware of new and innovative ground improvement ideas. If a new or innovative ground improvement method is to be used on an SCDOT project, approval must be first obtained from the GDS and the PCS/GDS. The approval process will consist of a minimum of engineering design, the desired outcome, construction methodology, and availability of construction experience/contractors to perform the specified type of work.

Ground improvement construction methods are used to improve poor/unsuitable subsurface soils and/or to improve the performance of embankments or structures. These methods are used when replacement of the in-situ soils is impractical because of physical limitations, environmental concerns, or is too costly. Ground improvement methodologies have the primary functions to:

- Increase bearing capacity, shear, or frictional strength,
- Increase density,
- Control deformations,
- Accelerate consolidation,
- Decrease imposed loads,
- Provide/increase lateral stability,
- Form seepage cutoffs or fill voids,
- Increase resistance to liquefaction, and
- Transfer embankment and/or ERS loads to more competent layers.

According to Ground Improvement Methods, "There are three strategies available to accomplish the above functions representing different approaches."

1. Increase shear strength, density, and/or decrease compressibility of foundation soil,
2. Use lightweight fills to significantly reduce the applied load on the foundation soil, and
3. Transfer the load to a more competent (deeper) foundation soil.

Ground Improvement Methods recommends a sequential design process that includes a sequence of evaluations that proceed from simple to more detailed. This process identifies the best method and is defined in Table 19-1.

Table 19-1, Ground Improvement Design Process (modified from Ground Improvement Methods – August 2006)

Step	Process
1	Identify potential poor ground conditions, including extent and type of negative impact
2	Identify or establish performance requirements (see Chapter 10)
3	Identify and assess any space or environmental constraints
4	Assessment of subsurface conditions – type, depth and extent of poor soil as well as groundwater table depth and assessment of shear strength and compressibility
5	Preliminary selection – takes into account performance criteria, limitations imposed by subsurface conditions, schedule and environmental constraints, and amount of improvement required (Table 19-2 should be used in this selection process)
6	Preliminary design
7	Comparison and selection – selection is based on performance, constructability, cost, and any other relevant project factors

Table 19-2, Ground Improvement Categories, Functions and Methods (modified from Ground Improvement Methods – August 2006)

Category	Function	Method
Consolidation	Accelerate consolidation and increase shear strength	1.) Prefabricated Vertical Drains 2.) Surcharge
Load Reduction	Reduce load on foundation and reduce settlement	1.) Geofoam 2.) Foamed Concrete 3.) Lightweight fill
Densification	Increase density, bearing capacity, and frictional strength of granular soils. Decrease settlement and increase resistance to liquefaction	1.) Vibro-Compaction 2.) Dynamic Compaction by falling weight impact
Reinforcement	In soft foundation soils, increases shear strength, resistance to liquefaction, and decreases compressibility	1.) Stone Columns
Deep Soil Mixing	Physio-chemical alteration of foundation soils to increase their tensile, compressive, and shear strength; to decrease settlement; and/or provide lateral stability and/or confinement	1.) Wet mixing methods 2.) Dry mixing methods
Grouting	To form fill voids, increase density, increase tensile, and compressive strength	1.) Permeation Grouting 2.) Compaction Grouting 3.) Jet Grouting
Load Transfer	Transfer load to deeper bearing layer	1.) Column Supported Embankment (CSE)

As indicated in Step 7, the cost of the ground improvement method must be considered in the selection process. Contact the PCS/GDS for cost information for ground improvements methods previously used by SCDOT. For ground improvement methods not previously used, every effort should be made to contact at least three contractors to obtain approximate pricing information.

According to Ground Improvement Methods, "The success of any ground improvement method is predicated on the implementation of a QA/QC program to verify that the desired foundation improvement level has been reached. These programs incorporate a combination of construction observations, in-situ testing and laboratory testing to evaluate the treated soil in the field. Details are provided in the *following Sections*." Italics in quotations indicate words added or changed to correspond to this Manual. The costs for the QA/QC program need to be added to the total cost of the ground improvement method.

19.2 PREFABRICATED VERTICAL DRAINS

Prefabricated vertical drains (PVDs), also commonly called wick drains, are used to accelerate consolidation of compressible cohesive soils to speed settlement and strength gain. The use of the term wick drains is a misnomer since water is not wicked out of the ground by the drains under capillary tension, but rather water flows from compressible clays under a pressure gradient induced by excess pore pressures associated with the placement of permanent fill and/or surcharge fill (see Figure 19-1).

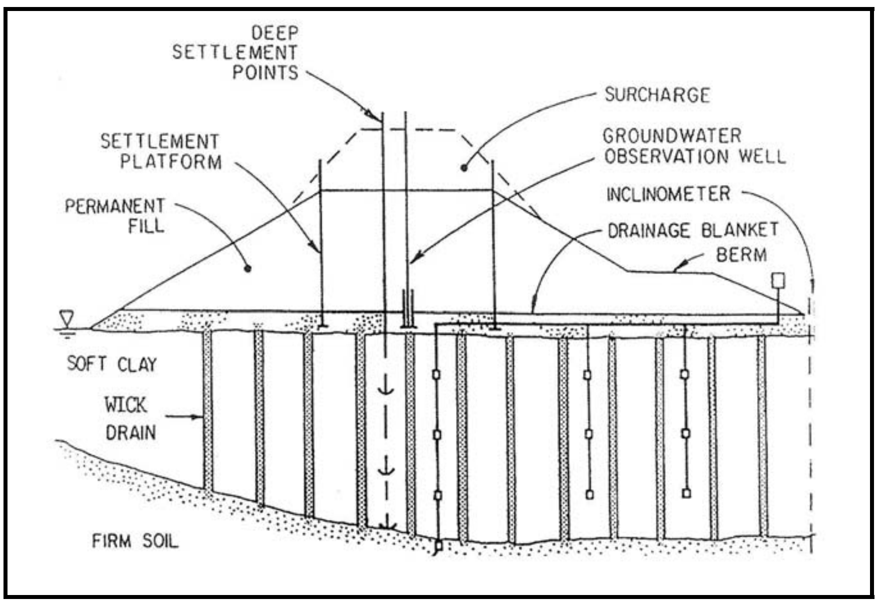

Figure 19-1, PVD Installation for a Highway Embankment (Ground Improvement Methods – August 2006)

PVDs have numerous advantages some of which include economy, installation speed, continuity of drain, and minimal displacement. Additional advantages are presented in Ground Improvement Methods, which should be consulted for greater details on this method. There are also some disadvantages to the PVDs which include greater quantities, no compressive strength, headroom limitations, and material must be properly handled and stored. It is noted that these disadvantages are in relation to the use of sand drains. The subsurface soils must be evaluated to determine the feasibility of using PVDs. The evaluation factors are provided below:

- Moderate to high compressibility
- Low permeability
- Full saturation
- Final embankment loads must exceed maximum preconsolidation stress (σ'_p or p'_c)
- Secondary compression must not be a major concern
- Low-to-moderate shear strength
- Soils normally to slightly overconsolidated (OCR < 1.5)

PVDs are thin plastic strips (about 1/8 inch thick by 4 inches wide) consisting of a rigid core sheathed in filter fabric. They have generally replaced sand drains and the design theories were adapted from sand drain design. To accelerate the rate of settlement, PVDs, are typically installed on a regular grid pattern, either triangular or rectangular, to reduce the flow distance for dissipation of excess pore water pressures associated with the placement of fill. Stone columns discussed later in this Chapter also can provide vertical drainage and similar methods can be applied to evaluate their effect on settlement rates.

19.2.1 Design Concepts

The primary purpose of PVDs is to reduce the length of the drainage path, thereby decreasing the time for settlement and strength gain to occur. Prior to selecting the use of PVDs, predictions of the amount and rate of settlement (see Chapter 17) both during and after construction are required. The amount of settlement must meet the performance criteria provided in Chapter 10. In addition, the stability of the embankment during the placement of the fill materials should also be ascertained. If the stability becomes questionable during construction, then vertical staging may be required. Chapter 17 discusses the stability of the embankment and vertical staging if required. Field testing (SPT, CPT and/or DMT) is required to determine if pre-drilling is necessary. The principle of PVD design is the selection of the type, spacing and length of the drains to accomplish the required performance limit (degree of consolidation) within a specified time.

According to Ground Improvement Methods, "The assumptions used in developing one dimensional consolidation theory were applied to the development of radial drainage theory related to vertical drains, which resulted in the following relationship between time, drain diameter, spacing, coefficient of consolidation and the average degree of desired consolidation."

$$\boldsymbol{t = \frac{D^2}{8C_h}\left(F(n) + F_S\right)\ln\left(\frac{1}{1-\overline{U_h}}\right)} \qquad \text{Equation 19-1}$$

Where,

t = Time required to achieve desired average degree of consolidation

$\overline{\boldsymbol{U_h}}$ = Average degree of consolidation due to horizontal drainage

D = Diameter of the cylinder of influence of the drain (drain influence zone)

C_h = Consolidation Coefficient for horizontal drainage

F(n) = Drain spacing factor (see Equation 19-2)

d = Equivalent circular drain diameter
F_S = Factor for soil disturbance

This equation does not include any consolidation due to vertical drainage. It is noted that the predicted settlement amounts and rates (discussed in Chapter 17) are based on vertical drainage.

$$\boldsymbol{F(n) = \ln\left(\frac{D}{d}\right) - 0.75} \qquad \text{Equation 19-2}$$

The following sections contain a discussion of each of these components.

19.2.1.1 Determination of F_S

Soil disturbance is typically ignored except for highly plastic (PI > 21), sensitive (S_t > 5) soils, where the Consolidation Coefficient for vertical drainage (C_v) has been accurately determined. For these soils an $F_S \approx 2$ should be used, otherwise use $F_S = 0$. Sample disturbance is more pronounced at drain spacings of less than 5 feet or by the use of large, thick anchor plates.

19.2.1.2 Determination of C_h

The horizontal Consolidation Coefficient (C_h) can be obtained only through laboratory consolidation testing of high quality samples. Even with high quality samples and testing, the results of the testing can be off by 50 percent of the actual values. Normally C_h is greater than C_v. A conservative approach is set C_h equal to C_v, without direct measurements of C_h. However, for design, C_h can be taken as 1.2 to 1.5 C_v, if no or only slight evidence of layering is evident in partially dried clay samples. If layering of silt and sand in discontinuous lenses is evident, C_h may be 2 to 4 C_v. The horizontal Consolidation Coefficient may be assessed in the field using CPT instrumentation and allowing for pore pressure dissipation.

19.2.1.3 Determination of d

The equivalent circular drain diameter (d) of a PVD has been determined using various methods. Diameters ranging from 1.6 to 5.5 inches have been used for the equivalent circular drain diameter, with the most common being 2.4 inches.

19.2.1.4 Determination of $\overline{U_h}$

The average degree of consolidation ($\overline{U_h}$) should meet the performance limit requirements of Chapter 10. Vertical consolidation can contribute significantly to the total amount of vertical movement and should be considered in the development of the degree of consolidation required.

19.2.1.5 Determination of D

According to <u>Ground Improvement Methods</u>, “When using an equilateral triangular pattern, *the diameter of the cylinder of influence (D)*, is 1.05 times the spacing between each drain. In a square pattern, D is 1.13 times the spacing between drains. Typically, to achieve approximately

90 percent consolidation in 3 to 4 months, designers often choose drain spacing between 3 to 5 feet in homogeneous clays, 4 to 6 feet in silty clays and 5 to 6-1/2 feet in coarser soils."

19.2.1.6 Determination of t

The time (t) is the duration required to achieve the desired average degree of consolidation ($\overline{U_h}$) for a diameter of the cylinder influence (D) and drain diameter (d). According to Ground Improvement Methods, "There are three basic variables that can be manipulated in order to achieve a desired result from Equation *19-1*. These variables are time, PVD spacing, and surcharge. *In order to increase the PVD spacing and reduce the number of PVDs installed, the surcharge can be increased to provide the same amount of consolidation over the same time period*." The addition of surcharge and keeping the PVD spacing the same has the effect of reducing the time for consolidation to occur. Typically, time is used as a constant (normally set to meet a specific construction schedule) and the amount of surcharge and the PVD spacing are used as variables.

19.2.1.7 Computer Software

Simple applications can be analyzed with hand calculations or with the use of a spreadsheet program to facilitate sensitivity studies. The computer program, FoSSA 1.0, can be used for analyses where the rate of loading becomes more complex and hand solutions become impractical.

A complete set of the design calculations prepared in accordance with this Chapter shall be provided. The determination of surcharge amounts and PVD spacing shall be fully documented with all design calculations. Submitted calculations (including computer input and output) shall include all assumptions used in the analysis. Computer generated designs made by software other than FHWA's FoSSA computer program shall require verification that the computer program's design methodology meets the requirements provided herein; this shall be accomplished by either:

1. Provide complete, legible, calculations that show the design procedure step-by-step for the surcharge and PVD spacing. Calculations may be computer generated provided that all input, equations, and assumptions used are shown clearly.

2. Provide a diskette with the input files and the full computer output of the FHWA sponsored computer program FoSSA (latest version). This software may be obtained at:

ADAMA Engineering, Inc.
33 The Horseshoe, Covered Bridge Farms
Newark, Delaware 19711, USA
Tel. (302) 368-3197, Fax (302) 731-1001

19.2.2 Earthquake Drains

Earthquake (EQ) drains are a subset of PVDs that are used to mitigate/limit the effects of seismically induced liquefaction. While PVDs are thin plastic strips consisting of a rigid core sheathed in filter fabric; EQ drains are perforated, corrugated plastic pipe placed in a filter fabric sock. Earthquake drains can range in size from 1-1/2 to 10 inches in diameter, but are more typically 4 to 6 inches in diameter. Earthquake drains are used to reduce the excess pore pressures generated by a seismic event that can lead to liquefaction in loose granular soils (see Chapter 13 for a discussion of liquefaction). The theoretical background for earthquake drains is presented in *FEQDrain: A Finite Element Computer Program for the Analysis of the Earthquake Generation and Dissipation of Pore Water Pressure in Layered Sand Deposits with Vertical Drains*.

EQ drains work by reducing the pore pressure ratio (r_u, see equation 19-3), to a level that prevents or limits the potential for liquefaction. Recent research on the applicability of EQ drains has indicated that some liquefaction induced settlement will still occur. Typically a r_u of 0.65 is used to determine the spacing of the drains. However, because of the uncertainties in the amount of liquefaction induced settlement, the effect of high fines content (i.e., percent passing the No. 200 greater than 5 percent), and the effect of high accelerations caused by earthquakes in South Carolina, the r_u shall be limited to 0.50. Using an r_u of this magnitude will cause the drain spacing to become smaller and potentially increasing the drain size.

$$r_u = \frac{\Delta u}{\sigma'_v} \qquad \text{Equation 19-3}$$

Where,

r_u = Pore pressure ratio

Δu = Change in pore pressure

σ'_v = Effective overburden pressure

19.2.3 Construction Considerations

PVDs are installed using equipment similar in size and appearance to pile driving equipment and or foundation drilling equipment. A typical installation rig for PVDss is shown in Figure 19-2. The contractor is required to submit an installation plan, shop drawings, material samples, and anchorage details. A minimum 12-inch thick layer of clean sand is necessary at the top of the PVDs to provide a drainage path for release of the excess pore pressures. In some applications it will be appropriate to install strip drains across the ground surface to provide horizontal drainage at the top of the PVDs. The drainage layer many times can be installed as a part of the working platform necessary to make the site accessible to PVD installation equipment.

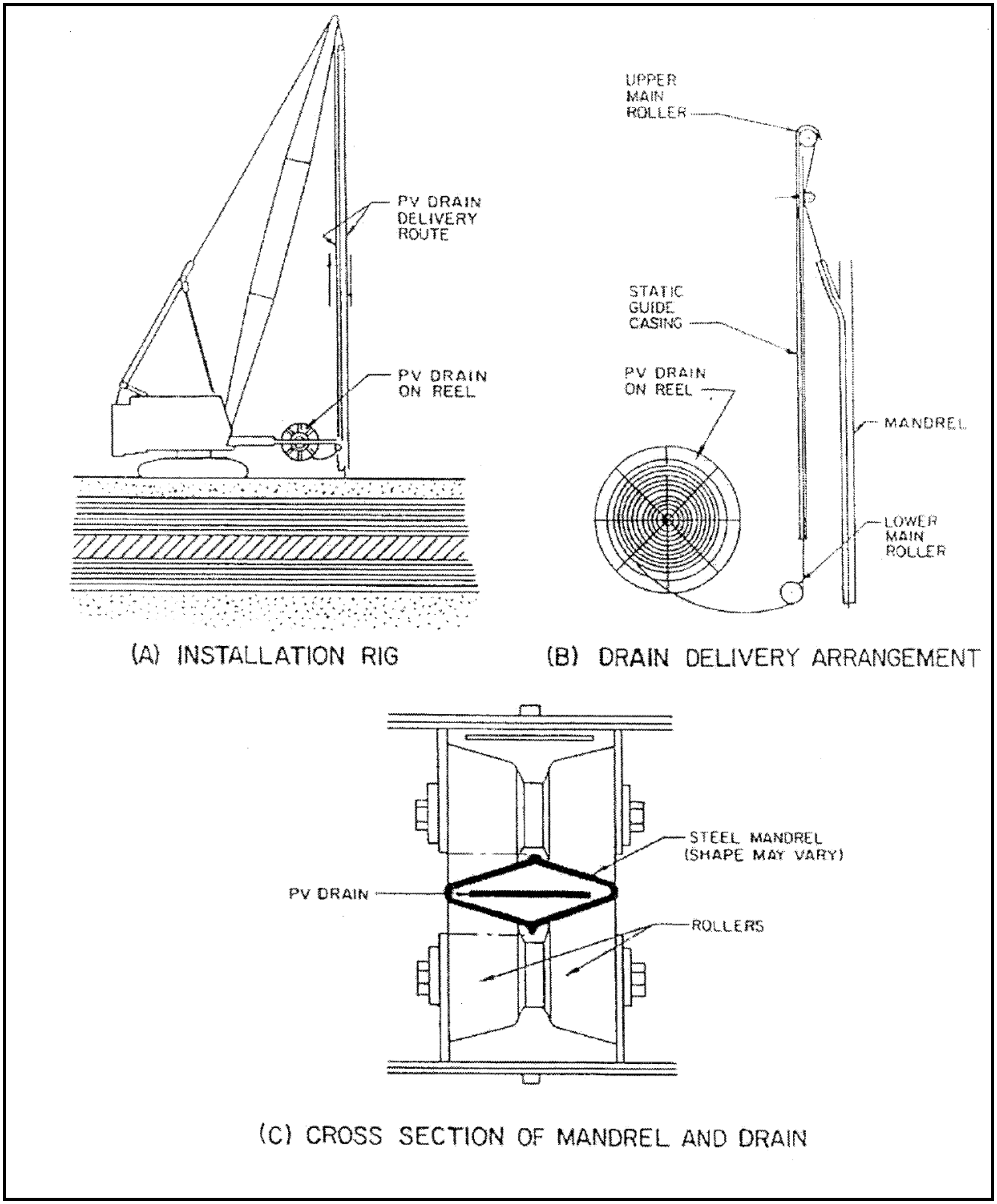

Figure 19-2, Crane Mounted Installation Rig
(Ground Improvement Methods – August 2006)

19.3 LIGHTWEIGHT FILL MATERIALS

Lightweight fill materials are used to limit settlement and increase stability through the use of materials with lower densities than conventional fill materials. Conventional fill materials (i.e., sand, silt and gravel) have densities that range from 115 to 140 pounds per cubic foot (pcf).

Lightweight fill materials can have densities ranging from 1 pcf for geofoam to 65 pcf for expanded clays and shales. In addition to reducing settlement and increasing stability, lightweight fill materials reduce the load applied to Earth Retaining Structures and increase an embankment's resistance to seismic loads by reducing the seismic inertial forces. Table 19-3 provides a list of lightweight fill materials used by SCDOT. Ground Improvement Methods provides additional lightweight materials; however, the use of these other lightweight fill materials, must be approved in writing by SCDOT (including the GDS, the PCS/GDS, and Office of Materials and Research (OMR))

Table 19-3, Lightweight Fill Materials
(modified from Ground Improvement Methods – August 2006)

Fill Material	Range of Density (pcf)	Range of Specific Gravity
Geofoam (EPS)	0.75 – 2.00	0.01 – 0.03
Foamed Concrete	20 – 60	0.3 – 0.8
Expanded Shale, Clay & Slate (ESCS)	37 – 65	0.6 – 1.0

19.3.1 General Applications and Limitations

19.3.1.1 Load Reduction

As indicated previously, one of the primary uses of lightweight fill is to reduce the load imposed on soft soils by normal weight fill materials. The use of lightweight fill materials reduces the driving forces, thereby increasing the overall global stability of the embankment or structure. A secondary effect of using lightweight fills is the reduction of the settlement under the imposed load. The amount of settlement reduction is directly proportional to the reduction in the load.

19.3.1.2 Shear Strength

According to Ground Improvement Methods:

> "Granular lightweight fills have an angle of shearing resistance similar to natural soils, while cemented lightweight fills are characterized by a compressive strength. These properties result in internal stability within the lightweight fills. In the case of an embankment over a weak foundation, the shearing surface will penetrate through the lightweight fill, and the shear strength developed within the lightweight fill deposits will tend to increase the overall global stability.

19.3.1.3 Compressibility

> Many lightweight fill materials, such as foamed concrete and ESCS have a compressibility and elasticity similar to natural soils and rock. Under static loading, the amount of internal compression within the fill will often be similar to that for conventional earth fill materials. Under dynamic loading, the resiliency of the lightweight materials will often be similar to the natural soils. Geofoam compressibility or stress strain behavior is generally linear to stress levels of about 0.5 percent. Beyond that, yielding occurs and the material is subject to time-dependent creep."

19.3.1.4 Lateral Pressures

According to Ground Improvement Methods, "The lateral earth pressure at any depth is a function of the vertical overburden pressure multiplied by the coefficient of earth pressure and then reduced by the cohesion of the deposit. In the case of lightweight fills such as foamed concrete or geofoam, the cohesion of the material is high and the densities are low. *Each* of these factors tend to *significantly* reduce the amount of lateral earth pressure that is transmitted to adjacent structures such as retaining walls, tunnels or pile foundations below bridge abutments."

19.3.1.5 Drainage

ESCS materials like most of the granular lightweight fill materials have good drainage characteristics. Good drainage is beneficial behind a retaining wall to eliminate hydrostatic pressures.

19.3.1.6 Construction in Adverse Weather

According to Ground Improvement Methods:

> "It is difficult, if not impossible, to place and compact conventional soils during extremely cold or wet weather. However, geofoam, ESCE and foam concrete, have been successfully installed in *inclement* weather.

19.3.1.7 Seismic Considerations

> In Japan, there have been case histories where a highway embankment constructed of geofoam did not fail in a severe earthquake, even though adjacent sections of a soil embankment did. The lower unit weight of the material results in lower inertial forces under seismic loading."

19.3.1.8 Limitations

Lightweight fill materials have limitations for use; however, these limitations can be overcome by proper evaluation, design, and construction techniques. The following list of limitations is obtained from Ground Improvement Methods:

- "*Availability of the materials*. Certain geographic areas may have an abundance of one type of lightweight fill material, but not of another. Unless the lightweight fill material is available *locally*, the transportation costs raise the price considerably, and make these materials non-competitive.
- *Construction Methods*. In general, all lightweight fill materials involve some special procedures with regard to handling, transportation, placement and compaction. Some lightweight fill materials could be difficult to place and handle. Foam concrete requires the use of specialized equipment at the site to introduce air and other additives into the mixture before placement.
- *Durability of the fill deposits*. Some lightweight fill materials (e.g., geofoam) must be protected to ensure longevity. Because geofoam is subject to deterioration from

hydrocarbon spills, a concrete slab or geomembranes are generally placed over the surface of the blocks.

- *Environmental concerns*. Some lightweight fill materials generate leachate as water passes through these deposits. Fortunately, design methods have been developed to minimize the amount of leachate, and, to date, these measures have worked satisfactorily. However, the additional costs of these measures should be considered during design.
- *Geothermal properties*. Most lightweight fill materials possess geothermal properties that are different than soil. This can lead to accelerated deterioration of flexible pavements and/or problems with differential icing of pavement surfaces due to an alteration of the heat balance at the earth's surface. Essentially, most lightweight fill materials act as thermal insulation, even though this is not an intended or desirable function. However, this can be effectively controlled by placing a suitable thickness (20-inch, minimum) of soil and/or paving materials over the surface of the lightweight fill material."

19.3.2 Geofoam

According to Ground Improvement Methods, "Geofoam is a generic term used to describe any foam material used in a geotechnical application. Geofoam includes expanded polystyrene (EPS), extruded polystyrene (XPS), and glassfoam (cellular glass). Geofoam was initially developed for insulation material to prevent frost from penetrating soils." Geofoam materials have the advantage of being not only lightweight, but also may be cut to any size of shape to fit the requirements of the project. NCHRP Project 24-11, Guidelines for Geofoam Applications in Embankment Projects, contains detailed design guidelines for the use of EPS geofoam in roadway embankments and bridge approaches. Geofoam is a lightweight fill material that has a specific compressive strength.

According to Ground Improvement Methods:

"The overall design process when using EPS geofoam is divided into three phases in order to consider the interaction between the three major components in the embankment.

- Design to preclude external (global) *in*stability of the embankment. This should include considerations for settlement, bearing capacity, and slope stability under the projected loading conditions.
- Design for internal stability within the embankment mass. The design must ensure that the geofoam mass can support the overlying pavement system without immediate and time dependent creep compression.
- Design of an appropriate pavement system for the subgrade provided by the underlying geofoam blocks.

External stability analyses generally follow traditional geotechnical procedures, although stress distribution must consider a non-homogeneous embankment. Stability analyses require modeling of undrained shear strength of geofoam, which presents some uncertainties. For shear strength, NCHRP 24-11 recommends the use of one-quarter of the compressive strength of the geofoam.

Internal stability analyses primarily consist of selecting an EPS geofoam type with adequate properties to support the overlying pavement system and the traffic load without excessive settlement of the surface. The current design approach is a deformation-based methodology using the total stress from all loads on the EPS blocks and elastic limit stress and the initial tangent modulus to evaluate load-induced deformations. Table *19-4* provides the minimum recommended values of elastic limit stress for various EPS densities." Table 19-5 summarizes the design parameters associated with the use of EPS geofoam.

Table 19-4, EPS Elastic Properties
(modified from Ground Improvement Methods – August 2006)

Material Designation	ASTM Designation	Dry Density each block (pcf)	Dry Density Test Specimen (pcf)	Elastic Limit Stress (psi)	Initial Tangent Young Modulus (psi)
EPS 40	I	1.00	0.90	5.8	580
EPS 50	VIII	1.25	1.15	7.2	725
EPS 70	II	1.50	1.35	10.1	1015
EPS 100	IX	2.00	1.80	14.5	1450

Table 19-5, EPS Geofoam Design Guidelines
(modified from Ground Improvement Methods – August 2006)

<table>
<tr><th colspan="4">Design Parameters</th></tr>
<tr><td>Density, dry</td><td>1 – 2 pcf</td><td>CBR</td><td>2 – 4</td></tr>
<tr><td>Compressive and Flexural Strength</td><td>6 – 14 psi[1]</td><td rowspan="2">Coefficient of Lateral Earth Pressure</td><td rowspan="2">Lateral pressures from adjacent soil mass may be reduced to a ratio of 0.1 of horizontal to vertical pressure</td></tr>
<tr><td>Modulus of Elasticity</td><td>580 – 1450 psi</td></tr>
<tr><th colspan="4">Environmental Considerations</th></tr>
<tr><td colspan="4">There are no known environmental concerns. No decay of the material occurs when placed in the ground.</td></tr>
<tr><th colspan="4">Design Considerations</th></tr>
<tr><td colspan="4">
EPS blocks will absorb water when placed in the ground. Blocks placed below water have resulted in densities of 4.8 – 6.4 pcf after 10 years, while blocks above the water had densities of 1.9 – 3.2 pcf for the same period. For settlement and stability analyses, use the highest densities to account for water absorption.
Buoyancy forces must be considered for blocks situated below the water table. Adequate cover should be provided to result in a resistance factor of 0.75 against uplift.
Because petroleum products will dissolve geofoam, a geomembrane or a reinforced concrete slab is used to cover blocks in roadways in case of accidental spills.
Differential icing potential of pavement, due to a cooler pavement surface above the EPS versus pavement above a soil only subgrade. Differential icing can be minimized by providing a sufficient thickness of soil between the EPS and top of the pavement surface.
Use side slopes flatter than or equal to 2H:1V and a minimum cover thickness of 1 foot. If a vertical face is needed, cover exposed face blocks with shotcrete or other material to provide long-term UV protection.
</td></tr>
</table>

[1]Varies with density

19.3.3 Foamed Concrete

Foamed concrete is created by introducing a preformed foam into a cement water slurry. The preformed foam is designed for concrete and creates a network of discrete air cells within the cement matrix. Sand and fly ash may be added to the mixture with the fly ash partially replacing a portion of the cement. After blending these materials to the specified density, the resulting slurry is pumped into place. Foamed concrete is unique for each application and is normally mixed on site. The quality of foamed concrete is monitored through the cast density. The compressive strength is directly related to the cast density of the mixture. Like geofoam, foamed concrete has a specific shear strength that is used in design.

According to Ground Improvement Methods:

> "Foam concrete is a liquid product that is practically self leveling, can be pumped over a distance as great as 3,300 feet, and will begin to harden between 2 to 6 hours after production. Design with this product is analogous to design with conventional concrete, using the typical average properties summarized in Table *19-7* for a range of cast densities. In computing buoyant stability, the low value of the volumetric mass should be used. For settlement calculations, the high value of the volumetric mass should be used including consideration for water absorption.
>
> *Table 19-6* summarizes key design considerations."

Table 19-6, Foamed Concrete Design Guidelines
(modified from Ground Improvement Methods – August 2006)

<table>
<tr><th colspan="4">Design Parameters</th></tr>
<tr><td>Density, dry</td><td>20 - 61 pcf</td><td>Freeze-thaw Resistance, 100 cycles</td><td>92 – 98 %[1]</td></tr>
<tr><td>Compressive Strength</td><td>45 - 360 psi[1]</td><td rowspan="2">Coefficient of Lateral Earth Pressure</td><td rowspan="2">Negligible for vertical loads applied directly over the foamed concrete. Lateral pressures from adjacent soil mass may be transmitted undiminished.</td></tr>
<tr><td>Water Absorption</td><td>1.4 – 15 psf[1]</td></tr>
<tr><th colspan="4">Environmental Considerations</th></tr>
<tr><td colspan="4">There are no known environmental concerns.</td></tr>
<tr><th colspan="4">Design Considerations</th></tr>
<tr><td colspan="4">• Buoyancy could be a problem if foamed concrete is placed below the water table and there is not sufficient vertical confinement.
• The lower compressive strength mixes are affected by freeze-thaw cycles. The product should be used below the zone of freezing or a higher compressive strength used. Densities greater than 37 pcf have reported excellent freeze-thaw resistance.
• There is some absorption of water into the voids, which could affect the density and compressive strength. Saturation by water should be prevented by construction of a drainage blanket and drains.</td></tr>
</table>

[1]Varies with density

Table 19-7, Typical Properties of Foam Concrete
(modified from Ground Improvement Methods – August 2006)

	Foam Concrete Type (Density)							Unit
Density, cast	25	31	37	44	50	56	62	pcf
Density, cured, 28 days	23	29	36	42	48	55	61	pcf
Density, oven dry[1]	19	22	27	31	37	43	47	pcf
Average Compressive Strength, 28 days[2]	70	140	260	360	450	550	650	psi
Average Compressive Strength, 56 days[2]	90	170	300	400	580	650	750	psi
Tensile strength, 28 days, normal[3]	7	14	26	36	45	55	65	psi
Flexural strength, 28 days, normal[4]	12	25	47	65	81	99	117	psi
Young's modulus E, 28 days, normal[3]	$4.4x10^4$	$9.4x10^4$	$1.7x10^5$	$2.4x10^5$	$3.2x10^5$	$4.2x10^5$	$5.4x10^5$	psi
Freeze/thaw resistance at 100 cycles[5]	92	94	97	97	98			%
Freeze/thaw resistance at 300 cycles[6]	81	84	86	87	91			%
Shrinkage, 1 year, prism	0.65	0.55	0.45	0.45	0.4	0.4	0.35	%
Shrinkage, 1 year, practical	0.15	0.13	0.12	0.12	0.12	0.11	0.11	%
Hydraulic conductivity K $p_{c=}2.5$ psi[7]	$9x10^{-8}$	$4x10^{-8}$	$1x10^{-8}$	$8x10^{-9}$	$4x10^{-9}$			m/s
Hydraulic conductivity K $p_{c=}18$ psi[7]	$4x10^{-8}$	$1x10^{-8}$	$5x10^{-9}$	$3x10^{-9}$	$1x10^{-9}$			m/s
Water absorption[8]	75	50	33	22	15	10	7	k/m^2
Diffusion resistance μ								
+ R.H. = 50% to R.H. = 100%	2.5	3.5	4	4.5	5.5	6	6.5	--
+ R.H. = 70% to R.H. = 100%	5	6	7	8	9	10	12	--
Heat conductivity λ								
+ Oven dry	0.09	0.10	0.12	0.14	0.17	0.20	0.23	W/m.K
+ at R.H. = 70%	0.11	0.13	0.15	0.18	0.22	0.26	0.30	W/m.K
+ at R.H. = 95%	0.14	0.17	0.20	0.23	0.27	0.31	0.35	W/m.K

[1] Density when cured to constant mass at105° Celsius.
[2] Actual properties depend on mix designs, raw materials used, and manufacturing methods.
[3] Bending tensile strength and dynamic E-modulus determined with 2-point symmetrical bending test.
[4] With 4-point bending test.
[5] E_{dyn} as % of E_0 after 100 cycles according to ASTM C666, procedure B, modified for longer freeze/thaw cycle due to insulating properties of foam concrete.
[6] E_{dyn} as % of E_0 after 300 cycles according to ASTM C666, procedure B.
[7] K-value determined according to ASTM D 5084, method B, "falling head," using cell pressures or confining pressures of $P_c = 2.5$ psi & 18 psi.
[8] Total absorption in kg through 1 m^2 of foam concrete during 10 years exposure of that area to a constant head of 1 meter water.
[9] Heat conductivity (W/m.K) measurements are done with a Showa Denko K.K; the minimum value is valid for a moisture content of 70%, and the maximum value is valid for a R.H. of

19.3.4 Expanded Shale, Clay & Slate

Expanded shale, clay and slate (ESCS) is a granular lightweight fill material. In other words, the strength of these materials is based on the interlock between individual particles, similar to sands and gravels. ESCS is a synthetic aggregate created from heating certain shales, clays and slates in a rotary kiln to temperatures in excess of 1,800°F. During this process the clay minerals montmorillonite, illite, and kaolinite become completely dehydrated and expand. Once completely dehydrated, these materials will not re-hydrate under atmospheric conditions; therefore, retaining the expanded shape. The materials are graded through a screening process and may have rounded, cubical or sub-angular particle shapes. These particles are durable, chemically inert and relatively insensitive to moisture; however, the particles will absorb and retain some water. ESCS materials can be expensive to manufacture, which has led to the use of these materials primarily as lightweight aggregate in structural concrete.

The design procedures using ESCS use conventional geotechnical methods associated with granular soils. Table 19-8 summarizes key design considerations.

Table 19-8, ESCS Design Guidelines
(modified from Ground Improvement Methods – August 2006)

<table>
<tr><th colspan="5">Design Parameters</th></tr>
<tr><td rowspan="2">Dry Density</td><td>Compacted</td><td>50 – 65 pcf</td><td rowspan="2">Permeability</td><td rowspan="2">High</td></tr>
<tr><td>Loose</td><td>40 – 54 pcf</td></tr>
<tr><td rowspan="2">Angle of Shearing Resistance</td><td>Compacted</td><td>37° – 44°</td><td rowspan="4">Grain Size Gradation</td><td rowspan="4">5 – 25 mm</td></tr>
<tr><td>Loose</td><td>35°</td></tr>
<tr><td rowspan="2">Coefficient of Subgrade Reaction</td><td>Compacted</td><td>140 – 155 pci</td></tr>
<tr><td>Loose</td><td>33 – 37 pci</td></tr>
<tr><th colspan="5">Environmental Considerations</th></tr>
<tr><td colspan="5">There are no known environmental concerns.</td></tr>
<tr><th colspan="5">Design Considerations</th></tr>
<tr><td colspan="5">The material will absorb some water after placement, when continually submerged. Samples compacted at a water content of 8.5 percent have been found after 1 year to have a water content of 28 percent. Over a longer period of time, the estimated long-term water content would be about 34 percent.
Side slopes of embankments should be covered with a minimum of 3 feet of soil cover.
Use side slopes of 1.5H:1V or flatter to confine the material and provide internal stability.
For calculating lateral earth pressures, use an angle of shearing resistance of 35°.</td></tr>
</table>

19.4 VIBRO-COMPACTION

Vibro-compaction is a ground improvement method that uses a specialized vibrating probe for in-situ densification of loose sands at depths beyond that which surface compaction equipment is inadequate (see Figure 19-3). The vibrator densifies loose granular, cohesionless soils using mechanical vibrations and water to overcome the in-situ effective stresses between the soil grains causing the grains to rearrange under the action of gravity into a denser state. The vibrations in the immediate vicinity of the vibrator induce liquefaction of saturated loose granular, cohesionless soils. Generally, vibro-compaction can be used to achieve the following results:

- Increased soil bearing capacity
- Reduced foundation settlements
- Increased resistance to liquefaction
    - Compaction to stabilize pile foundations driven through loose granular materials
    - Densification for abutments, piers and approach embankment foundations
- Increased shear strength
- Reduced permeability
- Filling of voids in treated areas

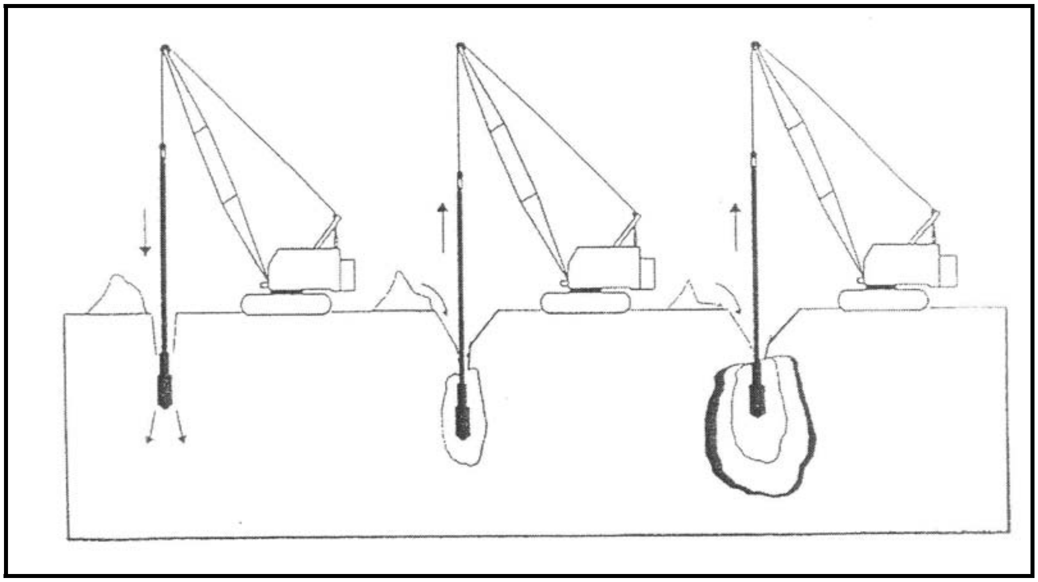

Figure 19-3, Vibro-Compaction
(Ground Improvement Methods – August 2006)

19.4.1 <u>Advantages and Disadvantages/Limitations</u>

19.4.1.1 Advantages

These advantages are described in, and come from, <u>Ground Improvement Methods</u>:

> "As an alternative to deep foundations, vibro-compaction is usually more economical and often results in significant time savings. Loads can be spread from the footing elevation, thus minimizing problems from lower, weak layers. Densifying the soils with vibro-compaction can considerably reduce the risk of seismically induced liquefaction. Vibro-compaction can also be cost-effective alternative to removal and replacement of poor load-bearing soils. The use of vibro-compaction allows the maximum improvement of granular soils to depths of up to 165 feet. The vibro-compaction system is effective both above and below the natural water level.

19.4.1.2 Disadvantages/Limitations

The major disadvantage of vibro-compaction is that it is effective only in granular, cohesionless soils. The realignment of the sand grains and, therefore, proper densification generally cannot be achieved when the granular soil contains more than 12 to 15 percent silt OR more than 2 percent clay. The maximum depth of 165 feet may be considered a disadvantage, but there are very few construction projects that will require densification to a greater depth.

Like all ground improvement techniques, a thorough soils investigation program is required. A more detailed soils analysis may be required for vibro-compaction than for a deep foundation project. This is because the vibro-compaction process utilizes the native soil to the full depth of treatment to achieve the end result. A comprehensive understanding of the total soil profile is therefore necessary. A vibro-compaction investigation will require continuous standard penetration tests (SPT) and/or cone penetrometer (CPT), as well as gradation tests to verify that the soils are suitable for vibro-compaction."

The potential environmental impact may be considered another disadvantage to vibro-compaction. The use of wet vibro-compaction, while efficient and used on the vast majority of projects, requires the use of water to jet the vibrator into the ground. The effluent from the jetting process requires at least temporary containment to allow any fine soil particles to settle out. Further, this method of ground improvement may not be acceptable if the existing subsurface environment, either soil or water is contaminated. If contamination is present, other ground improvement methods should be considered.

19.4.2 Design and Analysis

The design and analysis of vibro-compaction is based mainly on the grain-size distribution as shown in Figure 19-4. Soil compaction, as achieved in the vibro-compaction process through the rearrangement of soil particles, is not possible in cohesive, fine-grained soils. The cohesion between the particles prevents rearrangement and compaction from occurring. Soils on the coarse side of Zone B may be readily compacted using vibro-compaction. If the grain-size distribution curve falls in Zone C, it is advisable to backfill with gravel instead of sand during the compaction process. The use of gravel will improve the contact between the vibrator and the treated soil, drastically increasing compaction. Soils located partially or completely in Zone D are not suitable for vibro-compaction; however these soils (Zone D) are suitable for vibro-replacement (i.e. stone columns).

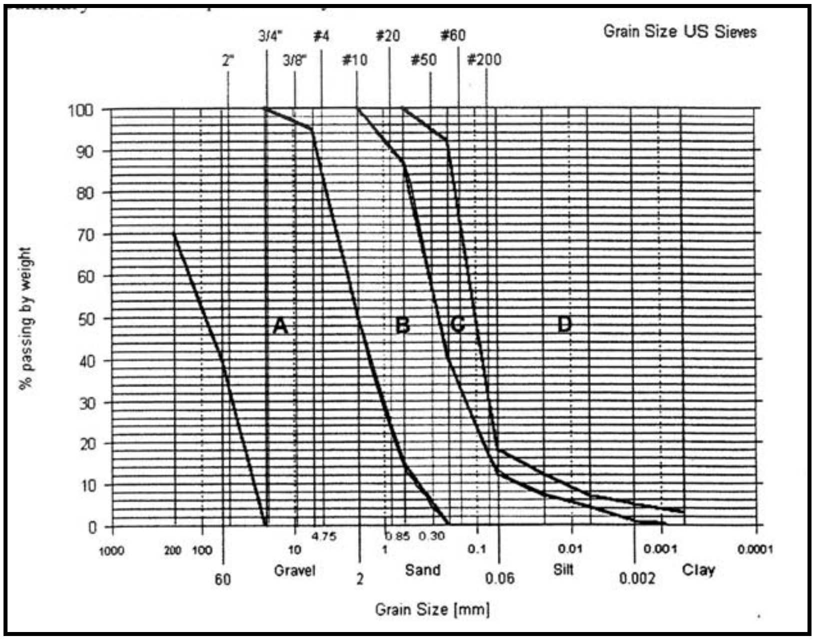

Figure 19-4, Soil Range Treatable by Vibro-Compaction (Ground Improvement Methods – August 2006)

As indicated previously, the vibrations induced by vibro-compaction cause the inter-granular forces acting between soil grains to reduce to zero allowing the soil particles to shift under the action of the vibrations and gravity into a more dense state. This more dense state has a reduced void ratio and correspondingly has a reduced compressibility and increase in the shearing resistance of the soil. The achievable reduction in void ratio depends on grain shape, soil composition (gradation), and vibration intensity. By controlling the advancement and withdrawal of the vibrator, a compact soil cylinder is formed. The diameter of the cylinder is based on the grain-size distribution, the initial soil density, and the vibrator characteristics. Typical vibrators have dynamic forces that range from 33,750 to 101,250 pounds with frequencies ranging from 1,800 to 2,300 revolutions per minute (rpm). For vibro-compactors operating at lower frequencies, better densification is usually produced. This is because low frequency vibrators usually have a higher amplitude, which translates into a greater compactive effort. Additionally, the natural frequency of most densifiable soils is closer to 1,500 rpm than to 3,000 rpm.

The increase in density of the granular soils causes a downward movement of the soil around the vibrator. This downward movement creates a conical depression at the ground surface. This depression requires constant filling with additional granular materials. A suitability number (S_N) is used to determine the suitability of a granular material as replacement material in vibro-compaction. The S_N is based on the settling rate of the backfill in water and experience. The S_N is determined using the following equation and the rating criteria are presented in Table 19-9. The backfill materials consist of sand or sand and gravel, with less than 10 percent by weight passing the #200 sieve and containing no clay.

$$S_N = 1.7\sqrt{\frac{3}{(D_{50})^2} + \frac{1}{(D_{20})^2} + \frac{1}{(D_{10})^2}}$$ Equation 19-4

Where,

D_{50} = Grain size diameters for 50 percent passing in millimeters
D_{20} = Grain size diameters for 20 percent passing in millimeters
D_{10} = Grain size diameters for 10 percent passing in millimeters

Table 19-9, Backfill Evaluation Criteria
(modified from Ground Improvement Methods – August 2006)

S_N	0 – 10	11 – 20	21 – 30	31 – 40	> 41
Rating	Excellent	Good	Fair	Poor	Unsuitable

19.4.2.1 Preliminary Design

The vibrator is hung from a crane cable or, in some instances; it is mounted to leads in a similar fashion as foundation drilling equipment. The vibrator penetrates under its self weight (or crowd of the machine if mounted in leads) and, at times, the action of water jets. The vibration and water imparted to the soils transforms the loose soils to a more dense state. Generally vibro-compaction would be used to achieve the following results on SCDOT projects:

- Increase soil bearing capacity
- Reduce immediate foundation settlement
- Increase resistance to liquefaction
- Increase shear strength

As indicated previously, the primary purpose of vibro-compaction is to increase the in-situ density of granular soils. Density is a measure of the unit weight of the soil; however, obtaining the unit weight of in-situ is extremely difficult. Therefore, the relative density (D_r) is used as measure of the increase or decrease in density of a soil. Relative density is defined in the following equation.

$$D_r = \frac{\gamma_n - \gamma_l}{\gamma_d - \gamma_l} \times \frac{\gamma_d}{\gamma_n} \times 100\% \qquad \text{Equation 19-5}$$

Where,

γ_n = Dry density of the soil in-situ

γ_l = Dry density of the soil in its loosest state

γ_d = Dry density of the soil in its densest state

Relative density has been correlated over the years in geotechnical practice. Chapter 6 also includes correlations of D_r versus SPT N-values. Table 19-10 provides the relationship between D_r and various field tests. Higher D_r equates to an increase in bearing capacity and a corresponding reduction in settlement. The resistance to liquefaction increases with increasing D_r and the active earth pressure on an ERS decreases while the passive earth pressure on an ERS increases. According to Ground Improvement Methods, "With vibro-compaction, the angle of internal friction is increased on average 5 to 10 degrees, resulting in much higher shear resistance."

Table 19-10, Penetration Resistance and Sand Properties (modified from Ground Improvement Methods – August 2006)

Description	Very Loose	Loose	Medium Dense	Dense	Very Dense
SPT N-values blows per foot	< 4	5 – 10	11 – 30	31 – 50	> 51
CPT Tip Resistance tsf	< 51	51 – 102	102 – 154	154 – 205	> 205
D_r %	< 15	16 – 35	36 – 65	66 – 85	86 - 100
Dry Unit Weight pcf	< 89	89 - 102	102 - 115	115 - 127	> 127
CSR[1]	< 0.04	0.04 – 0.12	0.12 – 0.33	0.33 – 0.40	-
Shear Wave Velocity, V_s fps	< 394	395 - 525	526 – 656	657 - 738	>739

[1]CSR – Cyclic Stress Ratio Causing Liquefaction

D_r is not only affected by the gradation of the soil, but also the area influenced by each compaction point. Figure 19-5 provides an example of an approximate relationship between D_r, soil type and treatment area for a specific vibrator. The increase in D_r is limited to 85 percent since, at this density, the improvement to the soil is enough to increase bearing and resistance to liquefaction and reduce settlement. It should be noted that a vibro-compaction contractor on an SCDOT project shall be required to present a similar type chart.

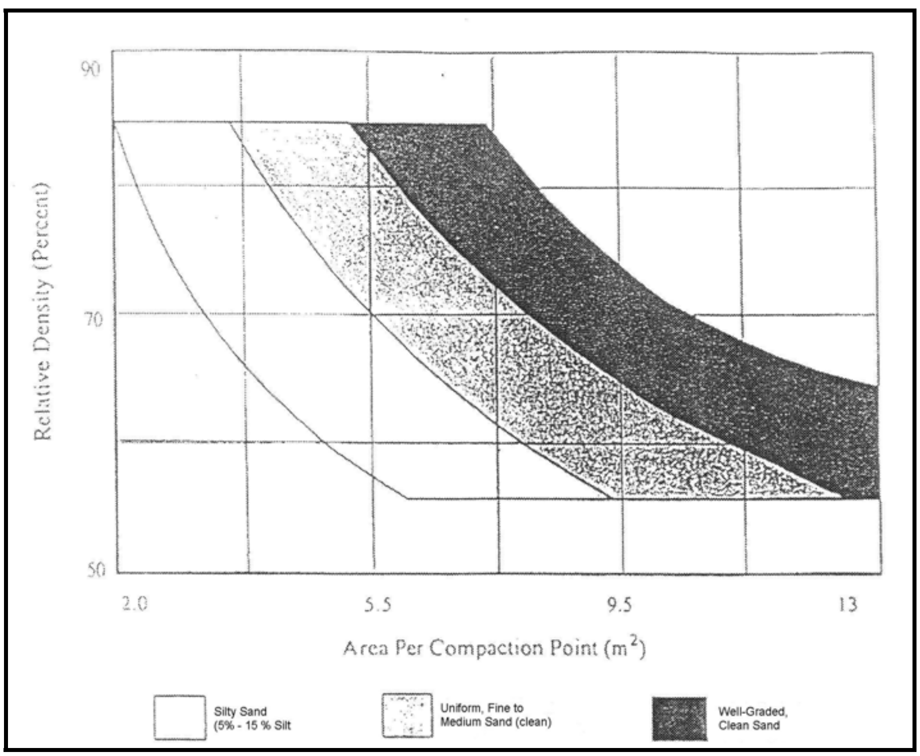

Figure 19-5, Variation of D_r with Tributary Area (Ground Improvement Methods – August 2006)

In South Carolina, vibro-compaction is primarily used to densify sites that have the potential for liquefaction (see Chapter 13). The improvement of the liquefiable soil should extend to the anticipated bottom of the liquefiable layer and should extend laterally to a distance at least equal to the depth of vibro-compaction as measured from the existing ground surface. Improvement for reducing lateral deformations of embankments is more effective when the foundation is treated in a zone between the crest and the toe of the embankment. Field performance suggests that the effect on structures will be minor when the supporting ground is improved to the "no liquefaction" side of the liquefaction potential curves (see Chapter 13).

According to Ground Improvement Methods:

> "A typical vibro-compaction program is designed with various probe spacing and patterns. The distance between compaction points is critical, as the density generally decreases as the distance from the probe increases. Stronger vibroprobes allow for wider spacing under the some soil conditions.
>
> The area compaction point pattern affects the densification. An equilateral triangular pattern is primarily used to compact large areas, since it is the most efficient pattern. The use of a square pattern instead of an equilateral triangular pattern requires 5 to 8 percent more points to achieve the same minimum densities in large areas.
>
> Given the in-situ soil gradation and relative density required, the spacing of compaction points can be determined. Figures *19-6 and 19-7* show typical area patterns and spacing for 80 percent relative density requirements. The spacing of the vibro-compaction points would be wider for lower relative density requirement."

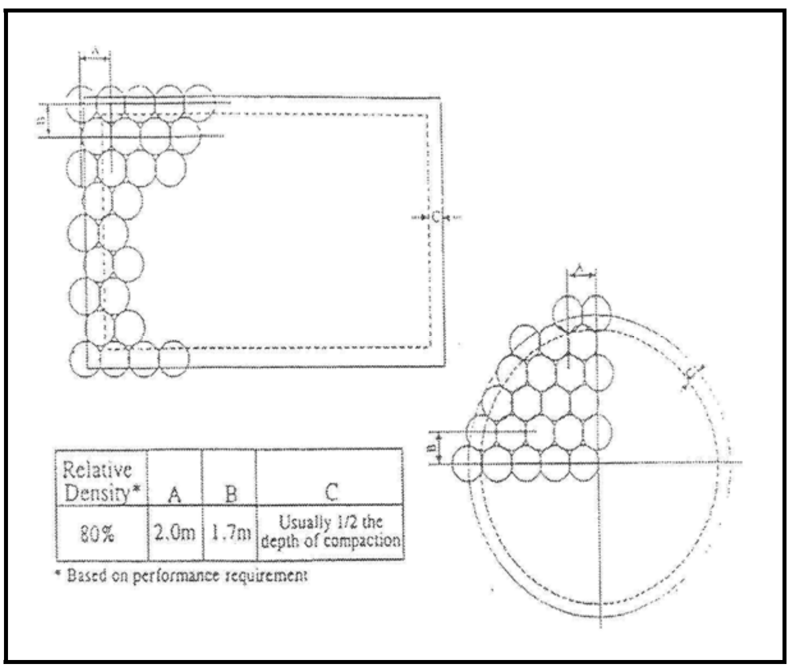

Figure 19-6, Typical Compaction Point Spacing for Area Layouts (Ground Improvement Methods – August 2006)

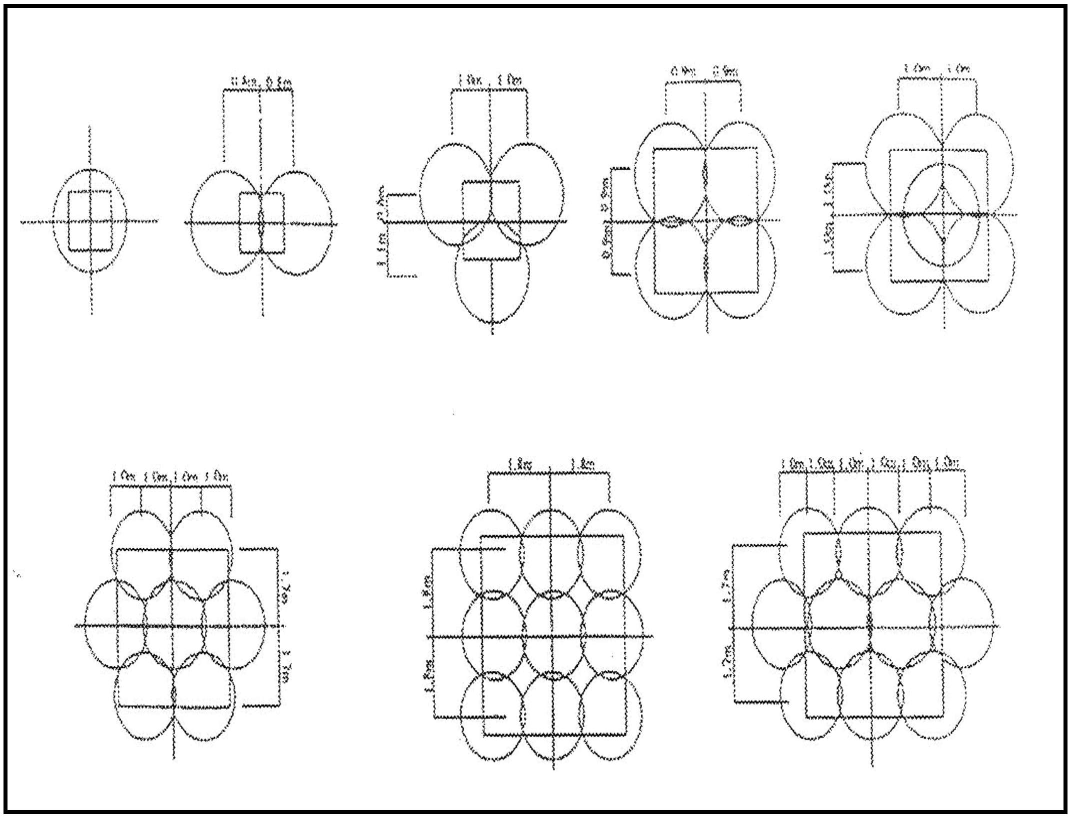

Figure 19-7, Typical Compaction Point Layouts for Column Footings (Ground Improvement Methods – August 2006)

19.5 STONE COLUMNS

Stone columns are constructed using vibratory methods similar to those methods used in vibro-compaction (see previous Section). The main differences is instead of using coarse-grained materials to simply fill the void created by the vibro-compaction, stone or other materials are placed to form a structural element (i.e., a column). Included in this Section along with stone columns are vibro-concrete columns (VCCs), geotextile-encased columns (GECs), and Geopier® Rammed Aggregate Pier^{TM} (Geopiers). Stone columns are constructed using either vibro-replacement or vibro-displacement. Table 19-11 provides definitions for both terms.

Table 19-11, Vibro-replacement and Vibro–displacement Definitions (Ground Improvement Methods – August 2006)

Vibro-replacement	Refers to the wet, top feed process in which jetting water is used to aid the penetration of the ground by the vibrator. Due to the jetting action, part of the in-situ soil is washed to the surface. This soil is then replaced by the backfill material.
Vibro-displacement	Refers to the dry, top or bottom feed process; almost no in-situ soil appears at the surface, but is displaced by the backfill material.

Stone columns are a natural progression from vibro-compaction and extended vibro-system applications beyond the relatively narrow application of densification of clean, granular soils as shown in Figure 19-8.

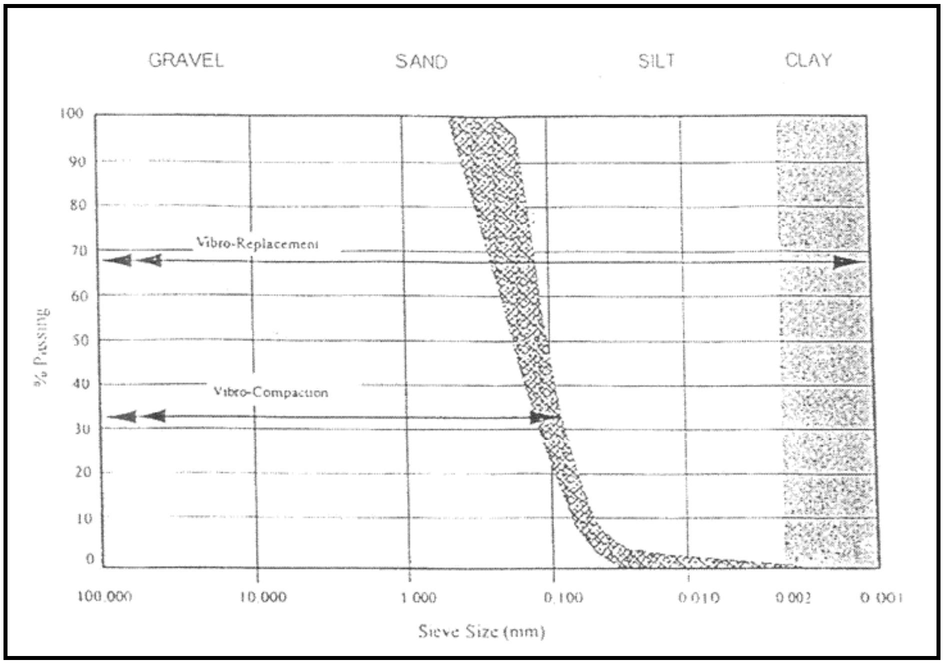

Figure 19-8, Applicable Grain-Size Distributions for Stone Columns (Ground Improvement Methods – August 2006)

As indicated previously, stone columns may be constructed using top or bottom feed methods (see Figures 19-9 and 19-10, respectively). The top feed method is a wet method and replaces the in-situ soil (i.e., vibro-replacement) with the stone column. In this method a high-pressure water jet is used to open a hole for the vibro-probe to follow into. Once the tip elevation is obtained the vibro-probe is retracted and stone is then placed into the hole from the top. The vibro-probe is then turned on and inserted into the stone to densify the stone, then the vibro-probe is retracted again and the process repeated until the stone column is formed. This method is used at sites with soft to firm soils with undrained shear strengths of 200 to 1,000 psf and a high groundwater table. When environmental impacts are anticipated, stone columns should be constructed using the vibro-displacement method. The vibro-displacement is a dry method that is either top or bottom feed. Using the oscillations of the vibrator in conjunction with the deadweight of the vibrator, air jetting and/or pre-augering, the vibrator is inserted into the ground without the use of jetting water. The top feed method can be used for short stone columns; however, for deeper columns and where the potential for hole collapse exists, the bottom feed method is used.

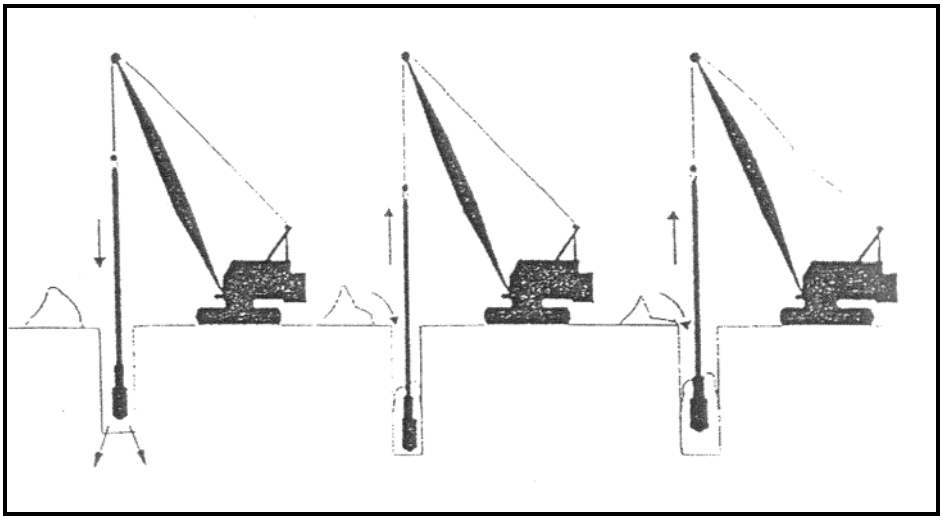

Figure 19-9, Top Feed Construction Method (Ground Improvement Methods – August 2006)

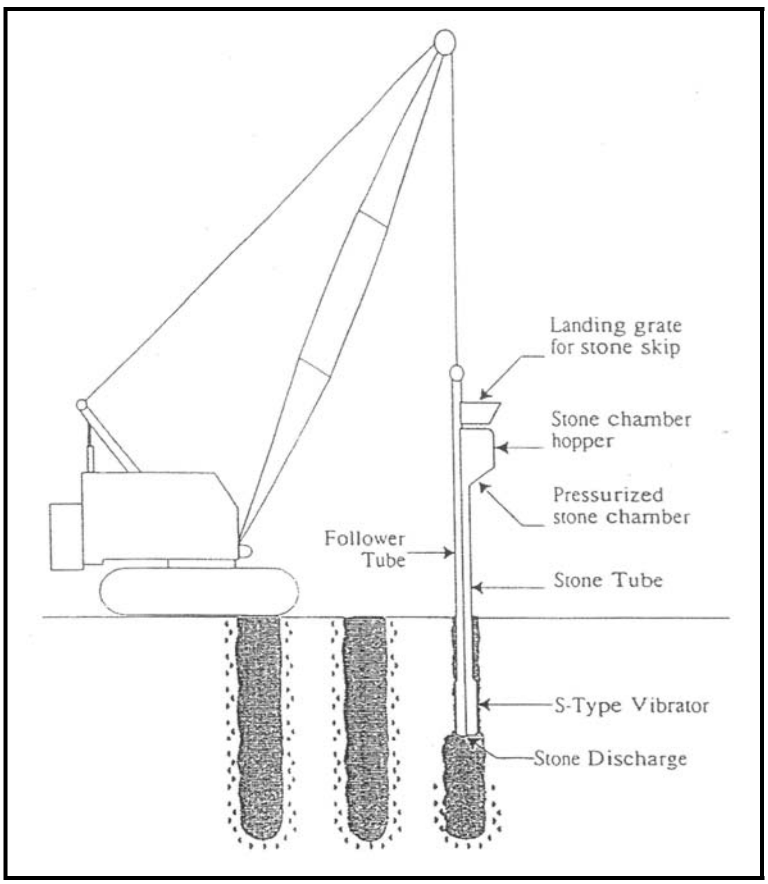

Figure 19-10, Bottom Feed Construction Method (Ground Improvement Methods – August 2006)

According to Ground Improvement Methods, "Since stone columns derive their strength and settlement characteristics from the surrounding soil, they do not perform well in very soft clay or peat with a thickness greater than the diameter of the column. Vibro-concrete columns *(VCCs)* were developed to treat these soils. Instead of feeding stone to the tip of the vibrator, concrete is pumped through an auxiliary tube to the bottom of the hole. This method can offer ground improvement advantages of the vibro-systems, with the load carrying characteristics of a deep foundation.

"The vibro-concrete column process employs a bottom feed vibrator that can penetrate the soils to a level suitable for bearing. Concrete is pumped through the vibrator assembly during initial withdrawal. The vibrator then repenetrates the concrete, displacing it into the surrounding soil to form a high-capacity, enlarged column base. The vibrator is then slowly withdrawn as concrete is pumped *and* maintained *at a* pressure to form a continuous shaft of concrete up to the ground level. At ground level, a slight mushrooming of the concrete column is constructed to assist the transfer of the applied loading into the vibro-concrete column (*see Figure 19-11*)."

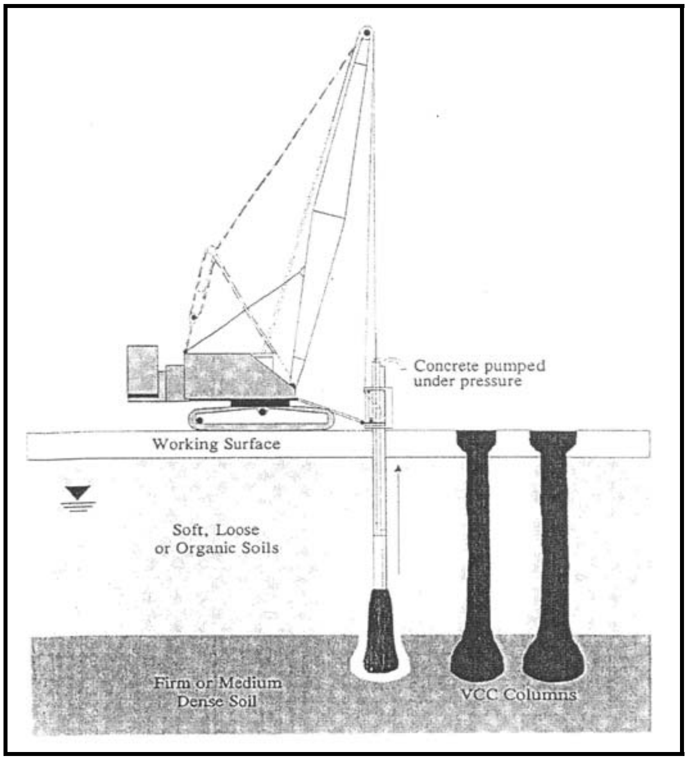

Figure 19-11, Vibro-Concrete Column
(Ground Improvement Methods – August 2006)

Geotextile-encased columns (GECs) consist of inserting continuous, seamless, high strength geotextile tubes into soft soil with a mandrel. The tube is then filled with either sand or fine gravel to form a column with a high bearing capacity. GECs typically have a diameter of 30 inches. GECs can be installed using either the replacement or the displacement methods. The replacement method consists of driving an open ended steel pipe pile to the bearing stratum. The soil within the pile is removed with an auger and the tube is inserted into the pile and then filled with sand or fine gravel. The displacement method use a steel pipe with two base flaps (the flaps close on contact with the ground surface) is vibrated to the bearing layer, displacing the soft soil. The geotextile casing is installed and filled with sand or fine gravel and the steel pipe pile is vibration extracted. During this process the sand or gravel within the geotextile is densified.

Geopier® Rammed Aggregate Pier™ (Geopiers) are a variant of stone columns, in that a 2- to 3-foot diameter hole is drilled into the foundation soil and gravel is added and then rammed into the foundation soils (see Figure 19-12). Geopiers typically extend to depths of 6 to 33 feet.

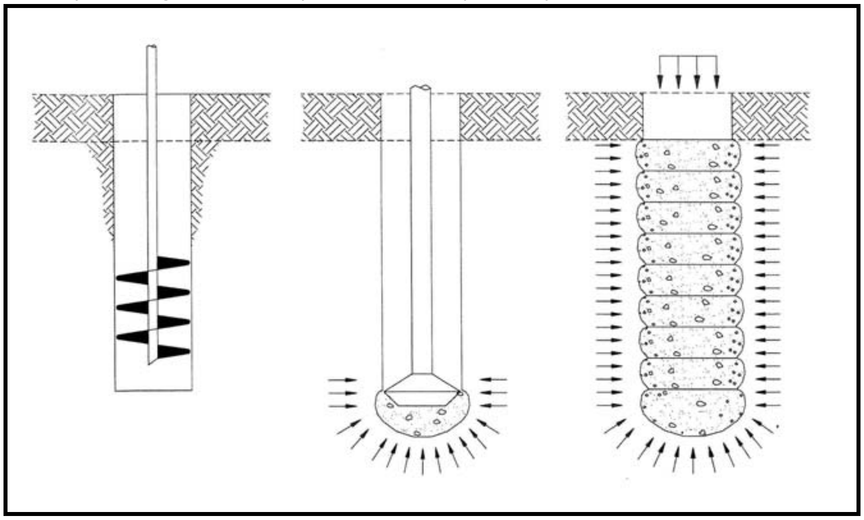

Figure 19-12, Geopier® Rammed Aggregate Pier™ (Ground Improvement Methods – August 2006)

Geopiers are most applicable in soft to stiff cohesive soils with undrained shear strengths ranging from 300 to 4,000 psf and in loose to medium dense silty and clayey sands. The soil must be stable without internal support (i.e., casing). The gravel is placed in relatively thin lifts with the first lift of gravel forming a bulb at the bottom of the pier, thus pre-stressing and pre-straining the soil beneath and around the bottom of the pier. The ramming process use a high-energy (250 to 650 kip-foot per foot) beveled tamper that both densifies the gravel and forces the gravel laterally into the sidewalls of the hole. This action increases the lateral stress in the surrounding soil, further stiffening the stabilized composite soil mass.

19.5.1 General Considerations

Stone columns can be used to improve the stability of slopes, increase bearing capacity, reduce total and differential settlements, and decrease the time for these settlements to occur, and to mitigate potential for liquefaction. Stone columns can be used to improve the stability of a slope by creating discrete zones of high strength material that will provide more resisting force along the potential failure surface. Stone columns can also increase the bearing capacity by transferring the load to a deeper, stronger layer and by densification of the in-situ soils through the use of vibro-displacement methods of installation. Further, stone columns can be used to reduce the amounts of total and differential settlement that a new embankment or a widened embankment would undergo without the improvement. The stone columns will also provide a conduit for the flow of ground water, thus decreasing the time for settlement to occur similarly to PVDs. Lastly, stone columns are used to mitigate the potential for liquefaction through densification of the in-situ materials and by providing pore pressure relief zones, because the stone column will have a greater hydraulic conductivity than the in-situ sands.

The advantages of stone columns are economy and technical feasibility to replace deep foundations with shallow foundations. Stone columns also provide a less expensive option to cut and replace, particularly on large sites with shallow groundwater. In developed areas where high-vibration methods such as dynamic compaction, deep blasting, or pile driving would have an impact on adjacent properties, low-vibration stone columns may provide a viable alternative to ground improvement. The use of stone columns could decrease the time required for construction by allowing construction to proceed immediately instead of waiting for the placement of surcharge. In areas that have a potential for liquefaction, the installation of stone columns can improve the cyclic resistance ratio (see Chapter 13). In addition, stone columns can provide vertical drainage and storage capacity to dissipate excess pore pressures induced by a seismic event. Geopiers have similar advantages to stone columns.

VCCs have the advantage of transferring loads similar to piles, while mobilizing the full ground improvement potential of a vibro-system. The installation of VCCs is a quiet process and induces minimal vibrations into the in-situ soils allowing for installation immediately adjacent to existing structures. Since this is a dry displacement process, there is no spoil to remove and no water requiring detention. VCCs have the additional advantage of being able to extend through thick very soft clays and organic materials.

According to Ground Improvement Methods, "The major advantage of GECs over stone columns is that they may be used in soft soils with undrained shear strengths as low as 25 psf. The geotextile provides the lateral constraint that the surrounding soils must provide for stone columns. GECs provide excellent vertical drainage, which may result in very rapid construction, due to the dissipation of pore water pressure."

The major disadvantage of stone columns is that stone columns are not effective in soils having thick layers of soft clays and organic materials. If the thickness is more than the diameter of the stone column, then stone columns may not be appropriate because the soft soils will not provide adequate lateral support of the stone column. In addition, stone column construction can be hampered by the presence of dense overburden, boulders, cobbles or other obstructions that may require pre-drilling prior to installation of the stone column. The major disadvantage of GECs and Geopiers is both methods rely on proprietary, patented technologies.

19.5.2 Feasibility

According to Ground Improvement Methods:

> "The degree of densification resulting from the installation of vibro-systems is a function of soil type, silt and clay content, soil plasticity, pre-densification relative densities, vibrator type, stone shape and durability, stone column area, column spacing, and compaction energy applied. Experience has shown that soils with less than fifteen percent passing the #200 (<0.074 mm) sieve, and clay contents less than two percent will densify due to the vibrations. Clayey soils do not react favorably to the vibrations, and the improvement in these soils is measured by the percent soil replaced and displaced by the stone columns, VCC, GEC, or Geopier.
>
> A generalized summary of the factors affecting the feasibility of stabilizing soft ground with stone columns is as follows:
>
> 1. The allowable design loading of a stone column should be relatively uniform and limited to a maximum of 112.5 kips per column, if sufficient lateral support by the in-situ soil can be developed.

2. The most significant improvement is likely to be obtained in compressible silts and clays occurring within 3.3 feet (10 m) of the surface and ranging in shear strength from 300 to 1000 psf.

3. Stone columns should not be used in highly sensitive soils *(see Chapter 7)*. Special care must be taken when using stone columns in soils containing organics and peat lenses or layers with undrained shear strength less than 200 psf. Because of the high compressibility and low strength of these materials, little lateral support may be developed and large vertical deflections of the columns may result. When the thickness of the organic layer is greater than one to two stone column diameters, the ability to develop consistent column diameters becomes questionable.

4. Ground improvement with stone column reduces settlements typically from thirty to fifty percent of the unimproved ground response and differential settlement from five to fifteen percent of unimproved soil response.

5. Stone columns have been used in clays having minimum (not average) undrained shear strengths as low as 150 psf. Due to the development of excessive resistance to penetration of the vibrator and economic considerations, a practical upper limit is in the range of undrained shear strength of 1,000 to 2,000 psf. Clays with greater shear strengths may, in fact, be strong enough to withstand the load without ground improvement.

6. Individual stone columns are typically designed for a bearing load of 20 to 30 tons per column. The ultimate capacity of a group of stone columns is predicted by estimating the ultimate capacity of a single column and multiplying that capacity by the number of columns in the group.

7. Stone columns have been used effectively to improve stability of slopes and embankments. The design is usually based on conventional slip circle or wedge analyses utilizing composite shear strengths.

8. *The following relationship is recommended to prevent piping of the soil surrounding the stone column:*

$$20D_{S15} < D_{G15} < 9D_{S85} \qquad \textbf{\textit{Equation 19-6}}$$

Where,

D_{S15} = Diameter of the surrounding soil passing 15 percent
D_{G15} = Diameter of stone (gravel) passing 15 percent
D_{S85} = Diameter of the surrounding soil passing 85 percent

A generalized summary of the factors affecting the feasibility of stabilizing soft ground with VCC follows:

1. The allowable design load for VCC is a function of the diameter of the column, the allowable strength of the concrete, and the strength of the bearing layer. Typical

column diameters range from 18 to 24 inches. Typical allowable design loads for VCC range from 75 to 100 tons.

2. VCC are typically used in very soft clay and organic soils.

3. Typical VCC lengths vary from 16 to 33 feet.

A generalized summary of the factors affecting the feasibility of stabilizing soft ground with GEC follows:

1. GEC may be installed in soft, compressible clays up to depths of approximately 33 feet. Typical column diameters range from 2 to 3 feet.

2. GEC allowable load capacity is 20 to 40 tons.

3. Settlement of GEC typically occurs during construction of embankment and may be up to 10 to 20 inches.

A generalized summary of the factors affecting the feasibility of stabilizing soft ground with Geopiers follows:

1. The allowable design load for a Geopier is typically in the range of 25 to 75 tons per pier, depending on the lateral confinement provided by the surrounding soils (i.e., undrained shear strength $\geq$ 300 psf for soft saturated clays and SPT N-value $\geq$ 1 *blow per foot* for cohesionless soils).

2. Geopiers have been used effectively in soft soils, provided that top-of-pier stresses are lower than those needed to initiate pier bulging into the soft soils.

3. The installation of Geopiers in soils that do not stand open during drilling (loose granular soils, very soft cohesive soils) often requires the use of temporary casing, which reduces the installation rate and increases the cost of the piers.

4. The maximum practical depth of Geopiers is limited to 33 feet."

19.5.3 Environmental Considerations

Vibro-replacement methods use water jets to create a hole for the vibro-probe. The jetted water can cause the fine portions of the in-situ soils to come to the ground surface. The fines laden soil has to be contained temporarily to allow for sediment deposition. The resulting deposited material has to be disposed of properly. Further, this method may also bring other contaminants to the ground surface, causing the treatment and proper disposal of not only the sediments, but also the water used for jetting. For these reasons, the use of dry vibro-displacement methods is preferred for the installation of stone columns.

19.5.4 Design Considerations

The design of stone columns is still an empirical process; however, general design guidelines have been developed and are provided below. Additional information may be obtained from the following references.

1. Design and Construction of Stone Columns, Volume I, FHWA/RD-83/026
2. "The Design of Vibro Replacement," *Ground Engineering*

For stone columns to adequately perform, the soils surrounding the columns must provide sufficient lateral support to prevent bulging failures. In addition, the columns should terminate in a dense formation to prevent bearing failures. Stone columns are typically stiffer than the materials that surround the columns; therefore, the columns will settle less and will carry a larger portion of the applied load. The applied load is transferred between columns through soil arching. Ultimately equilibrium is reached when sufficient load has been transferred to the columns to prevent further settlement of the surrounding soils. In stability and bearing analyses, composite shear strength of the soil-stone column matrix is used. The composite shear strength is based on the shear strength of the in-situ soils, the shear strength of the stone materials, the area replacement, and stress ratios.

19.5.4.1 Unit Cell Concept

According to Ground Improvement Methods, "For purposes of settlement and stability analyses, it is convenient to associate the tributary area of soil surrounding each stone column with the column illustrated in Figures *19-13* and *19-14*. Although the tributary area forms a regular hexagon about the stone column, it can be closely approximated as an equivalent circle having the same total area. The resulting equivalent cylinder of material having a diameter (D_e) enclosing the tributary soil and one stone column is known as the "unit cell". The stone column is concentric to the exterior boundary of the unit cell.

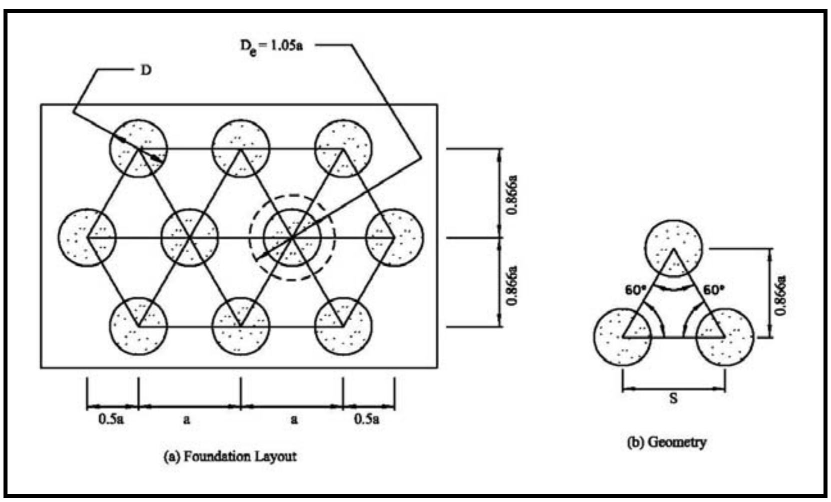

Figure 19-13, Stone Column Equilateral Triangular Pattern (Ground Improvement Methods – August 2006)

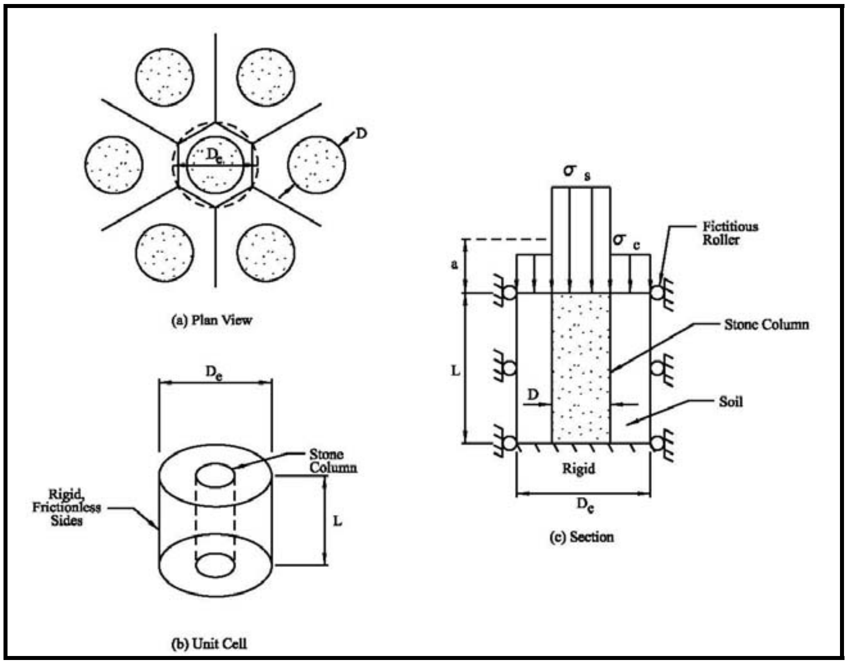

Figure 19-14, Unit Cell Idealization
(Ground Improvement Methods – August 2006)

19.5.4.2 Area Replacement Ratio

The Area Replacement Ratio (α_s) defines the area of the soil replaced by the stone column as a function of the tributary area of the unit cell to the area of the stone column. The more soil replaced by the stone column, the greater the effect on performance. Typical values of α_s range from 0.10 to 0.40.

$$\alpha_s = \frac{A_s}{A} \qquad \text{Equation 19-7}$$

$$a_s = \frac{1}{\alpha_s} = \frac{A}{A_s} \qquad \text{Equation 19-8}$$

Where,

α_s = Area replacement ratio
A_s = Area of the stone column
A = Total area within the unit cell
a_s = Area improvement ratio

19.5.4.3 Spacing and Diameter

According to Ground Improvement Methods, "Stone column diameters vary between 1.5 and 4.0 feet, but are typically in the range of 3.0 to 3.5 feet for the dry method *of installation*, and somewhat larger for the wet method *of installation*.

"Triangular, square or rectangular grid patterns are used with center-to-center column spacing of 5.0 to 11.5 feet. For footing support, *the stone columns* are installed in rows or clusters. For both footing or wide area support, *the stone columns* may extend beyond the loaded area."

19.5.4.4 Stress Ratio

The transfer of the applied load to the stone columns from the in-situ soils depends on the relative stiffness of the stone column to the in-situ soils, as well as the spacing and diameter of the stone columns. Because the stone columns and the in-situ soils deflect (strain) approximately equally, the stone columns must be carrying a greater portion of the load (stress) than the in-situ soils. This concept has also been called the equal strain assumption. This concept has been proven by both field measurements, as well as finite element analysis. The relationship between the stress in the stone column and the stress in the in-situ soil is defined in the following equation:

$$\boldsymbol{n} = \frac{\sigma_{\boldsymbol{s}}}{\sigma_{\boldsymbol{c}}} \qquad \text{Equation 19-9}$$

Where,

n = Stress ratio or stress concentration

σ_s = Stress in the stone column

σ_c = Stress in the surrounding soil

Measured values of n have generally been between 2.0 and 5.0. The theory indicates that n should increase with time. A high n-value (3 to 4) may be required in very weak soils and the column spacing is tight. Lower values of n (2 to 2.5) are required when the surrounding soil is stronger and the column spacing is wider. For preliminary design, a conservative n-value of 2.5 should be assumed.

Equilibrium of vertical forces for a given α_s is provided by the following equation.

$$\boldsymbol{q} = \sigma_{\boldsymbol{s}}\alpha_{\boldsymbol{s}} + \sigma_{\boldsymbol{c}}(\mathbf{1} - \alpha_{\boldsymbol{s}}) \qquad \text{Equation 19-10}$$

Where,

q = Average stress on the unit cell

The stresses in the stone column and the surrounding soil in the unit cell can be determined by rearranging the above equation.

$$\sigma_{\boldsymbol{c}} = \frac{\boldsymbol{q}}{[\mathbf{1} + (\boldsymbol{n} - \mathbf{1})\alpha_{\boldsymbol{s}}]} \qquad \text{Equation 19-11}$$

$$\sigma_{\boldsymbol{s}} = \frac{\boldsymbol{nq}}{[\mathbf{1} + (\boldsymbol{n} - \mathbf{1})\alpha_{\boldsymbol{s}}]} \qquad \text{Equation 19-12}$$

19.5.5 <u>Verification</u>

According to <u>Ground Improvement Methods</u>, "In-situ testing to evaluate the effect of the stone column construction on the native cohesive soil can be also specified. However, the specified test method should be selected on the basis of its ability to measure changes in lateral pressure in cohesive soils. The electric cone penetrometer *test* (CPT), the flat plate dilatometer *test* (DMT) and the pressuremeter *test* (PMT) appear to provide the best means for measuring the change, if any, in lateral stress due to stone column construction."

19.6 DYNAMIC COMPACTION

Dynamic compaction is the process of ground improvement using weights dropped from a height resulting in the application of high energy levels to the in-situ soil resulting in improvement of the soil. Typically, the weight (called a tamper) ranges from 11 to 39.6 kips and is dropped from heights of 30 to 100 feet. Dynamic compaction can typically be performed using conventional construction equipment as long as the crane has a free spool attached to allow the cable to unwind with minimal friction. The depth of improvement generally ranges from 10 to 36 feet for light- and heavy-energy applications, respectively. The light-energy applications consist of low weights and low drop heights, while heavy-energy applications consist of heavy weights dropped from high heights. Figure 19-15 provides a schematic of dynamic compaction.

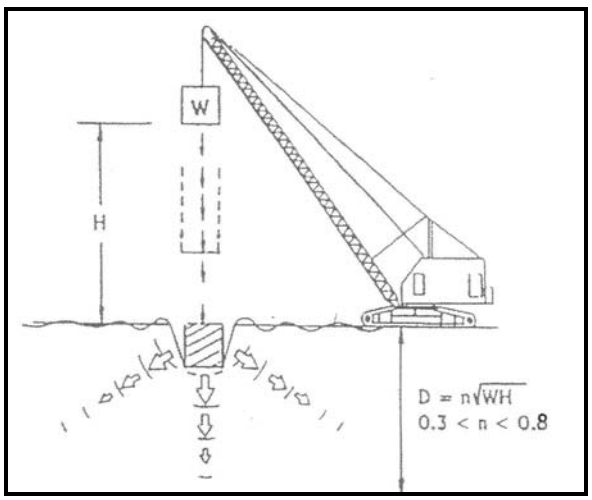

Figure 19-15, Dynamic Compaction Schematic
(Ground Improvement Methods – August 2006)

19.6.1 <u>Analysis</u>

Dynamic compaction is used to densify natural and fill deposits to improve the soil properties and performance of the subgrade soils. The primary uses of dynamic compaction are:

- Densification of loose deposits
- Collapse of large voids
- Related applications

Dynamic compaction is used to densify loose deposits of soil by reducing the void ratio. This ground improvement method is used for pervious, granular soils (Zone 1 - sands, gravels and non-plastic silts) that meet the gradation, permeability (hydraulic conductivity) and plasticity shown in Figure 19-16. For saturated Zone 1 soils, the induced excess pore pressures from dynamic compaction cause the soil particles to lose point-to-point contact (i.e. liquefy). Following dissipation of these excess pore pressures, the soil grains settle into a more dense structure. Besides permeability, the degree of saturation, length of the drainage path, and the soil stratigraphy also affect the effectiveness of dynamic compaction. The degree of saturation is related to the position of the groundwater table. For soils located above the groundwater table, the results of dynamic compaction are immediate, while time is required to allow pore pressure dissipation of soils below the water table. Dense or hard layers near the ground surface can limit the effect of dynamic compaction on deeper soils.

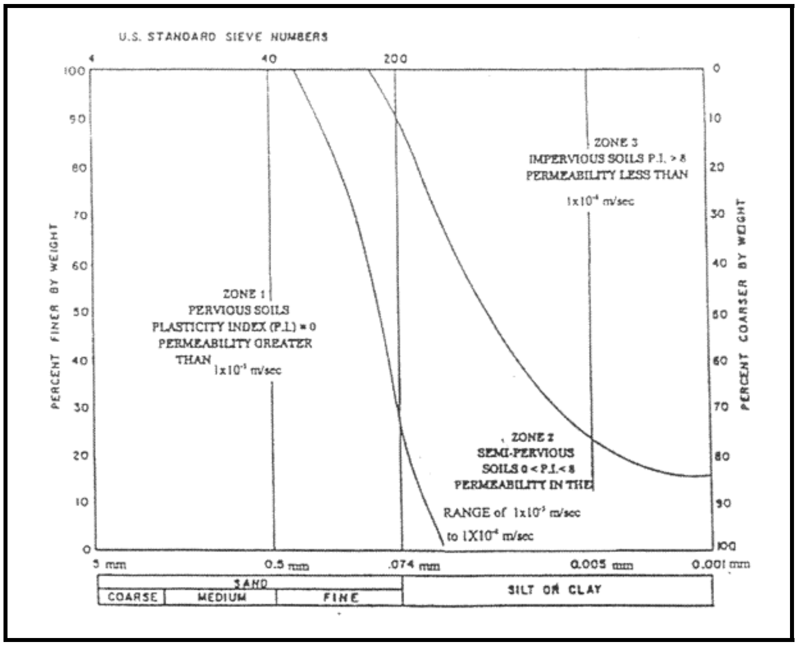

Figure 19-16, Soil Grouping for Dynamic Compaction (Ground Improvement Methods – August 2006)

Using a phase diagram, the results of multiple dynamic compaction passes verify the reduction in void ratio and the resulting densification of the subgrade soils (see Figure 19-17). It should be noted that while the void ratio decreases, the volume of the solids does not change.

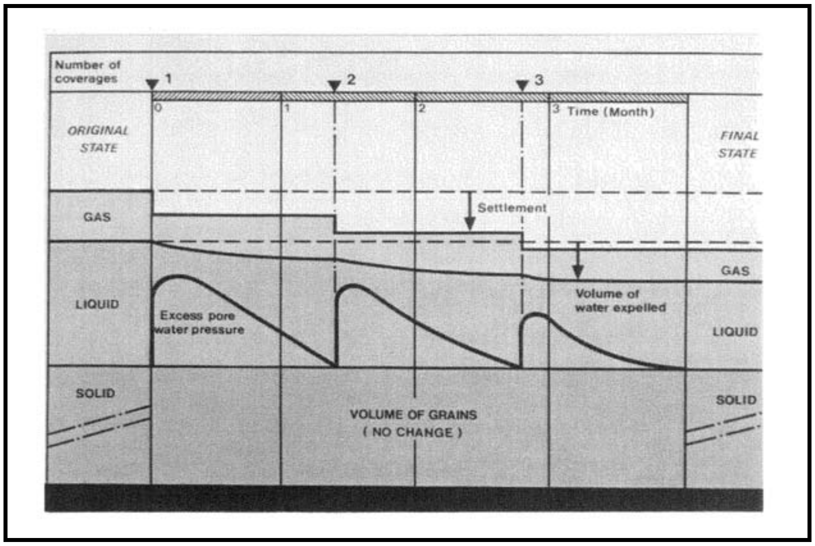

Figure 19-17, Dynamic Compaction Phase Diagram (Ground Improvement Methods – August 2006)

The soils indicated in Zone 3 (Figure 19-16) are typically impervious, plastic, fine-grained soils. The use of dynamic compaction is not recommended for these soils. The soils located in Zone 2 may be improved using dynamic compaction; however, multiple passes of the tamper will be required. In addition, additional time will be required between each pass to allow for the dissipation of excess pore pressures.

Large voids in natural or fill deposits can be collapsed using dynamic compaction depending on the depth to the void and the weight and drop of the tamper. Dynamic compaction can be used to improve fill materials of unknown compactive effort. In addition, dynamic compaction is also used to compact construction debris and solid waste materials that may be located within the Right-of-Way. Using dynamic compaction on construction debris and solid waste materials will improve the density of the material and may result in not having to remove and properly dispose of these materials.

According to Ground Improvement Methods, "In weak saturated soils relatively deep craters (> 5 feet) can develop. If these craters are filled with coarse granular materials and supplemental energy applied, the granular material will be driven into the weak deposit. This type of improvement is strictly speaking not dynamic compaction and is called dynamic replacement. The dynamic compaction equipment is used to produce the improvement, so this procedure is a related form of ground improvement. The depth of improvement is generally less than about 10 to 13 feet."

19.6.1.1 Advantages

Dynamic compaction has many advantages which are listed below:

- The tamper can be used as a probing, as well as a correcting, tool. Dropping the tamper can identify areas of loose soil or voids (deeper crater). This identification allows real time adjustments to the dynamic compaction program.

- Densification of soils can be observed as compaction proceeds. After several passes, the depth of the craters should become shallower indicating densification of the underlying soils.
- Dynamic compaction can be used on sites that have heterogeneous deposits (i.e., boulders, loose fills, construction debris, and solid waste).
- Dynamic compaction results in a bearing stratum that is more uniform after compaction, resulting in uniform compressibility, minimizing differential settlements.
- Densification can be achieved below the water table, eliminating costly dewatering.
- Standard construction equipment can be used for dynamic compaction with the exception of very heavy tampers and high drop heights. Very heavy tampers and high drop heights will require specialty contractors.
- Dynamic compaction can be performed in inclement weather, provided precautions are taken to avoid water accumulation in the craters.

19.6.1.2 Disadvantages

Dynamic compaction has the following disadvantages:

- Ground vibrations induced by dynamic compaction can travel significant distances from the point of impact, thus limiting the use of dynamic compaction to light weight tampers and low drop heights in urban environments.
- The groundwater table should be more than 6.5 feet below the existing ground surface to prevent softening of the surface soils and to limit the potential of the tamper sticking in the soft ground.
- A working platform may be required above very loose deposits. The working platform also functions to reduce the penetration of the tamper. The cost of the working platform can add significant costs to the project.
- Large lateral displacements (1 to 3 inches) have been measured at distances of 20 feet from the point of impact by tampers weighing 33 to 66 kips. Any buried structures or utilities within this zone of influence could be damaged or displaced.

19.6.1.3 Environmental Considerations

As indicated previously the vibrations created by dynamic compaction can have an adverse effect on adjoining properties. According to Ground Improvement Methods, "The U.S. Bureau of Mines has found that building damage is related to particle velocity. Figure *19-18* was developed by the Bureau based on experiences with damage measurements made in residential construction from blast-induced vibrations. The limiting particle velocity depends upon the frequency of the wave form. Normally, dynamic compaction results in frequencies of 5 to 12 *Hertz* (Hz). Using Figure *19-18* as a guide, this would limit peak particle velocities to values of ½-inch per second for older residences with plaster walls and ¾ inches per second for more modern constructions with drywall. Peak particle velocities that exceed the values given in Figure *19-18* do not mean damage will occur. Rather, these values are the lower threshold beyond which cracking of plaster or drywall may occur.

"Data generated by the U.S. Bureau of Mines indicate that minor damage occurs when the particle velocity exceeds 2 inches per second (51 mm/sec), and major damage occurs when the particle velocity exceeds about 7-1/2 inches per second (190-1/2mm/sec). Thus, keeping the particle velocity less than about ½ to ¾ inches per second should be a reasonably conservative value to minimize damage."

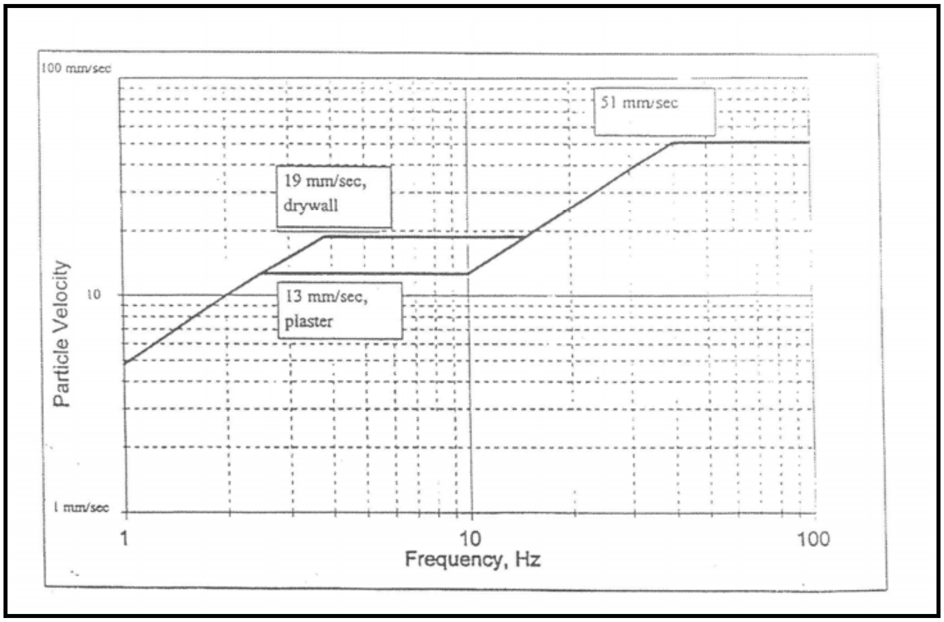

Figure 19-18, Safe Level of Vibrations for Houses (Ground Improvement Methods – August 2006)

Seismographs are typically used to measure ground velocities caused by dynamic compaction. Typically, a base line reading is obtained prior to commencing operations to obtain the level of ambient background vibrations. The readings during production operations are obtained from seismographs on adjacent structures or at the construction limits. However, prior to dynamic compaction production operations, an estimate of the particle velocity to be generated is required. Figure 19-19 can be used for planning purposes.

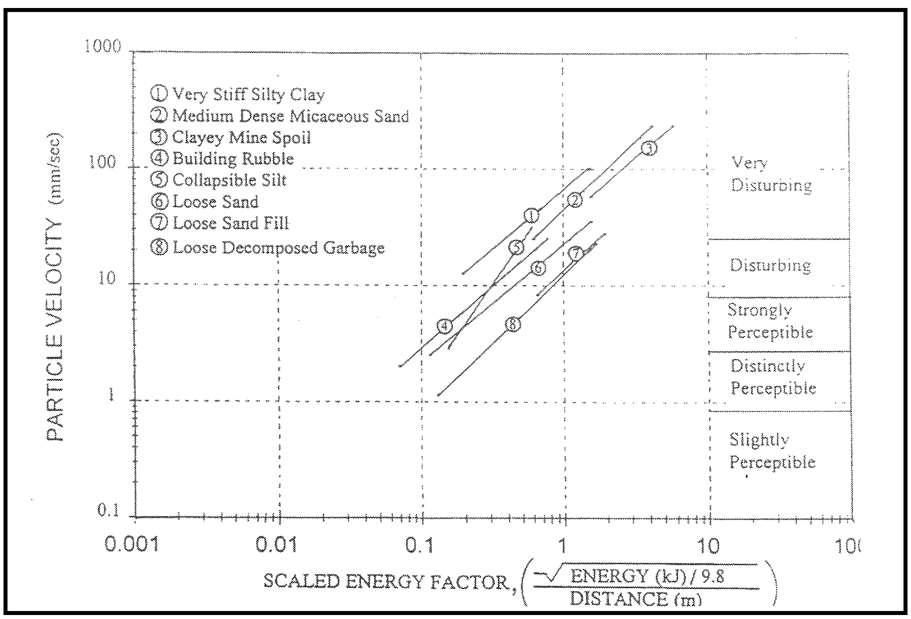

Figure 19-19, Scaled Energy Factor vs Particle Velocity (Ground Improvement Methods – August 2006)

kJ = 737.5 ft-pounds; m = .3048 ft; mm = 0.0394 inches

If the estimated particle velocity exceeds the project requirements, then, either the weight of the tamper is reduced or the drop height is lowered. Ground vibrations on the order of ½ to ¾ inches per second are perceptible to humans. Even though these vibrations should not cause damage, vibrations of this magnitude can lead to complaints. Educating the adjacent property owners to the potential impacts of the ground vibrations should be performed.

Dynamic compaction can lead to lateral soil movement. Measurements and observations from other projects has indicated tampers ranging from 33 to 66 kips should not be used within 20 to 30 feet of any buried structure, if movements can cause damage to the structure. In addition, flying debris can occur following impact of the tamper. To avoid flying debris, a safe working distance should be established from the point of impact. Dynamic compaction has an effective depth limitation of approximately 36 feet.

19.6.2 Design

After determining if dynamic compaction is a viable ground improvement method, the next step is to develop a more specific ground improvement plan including the following:

- Determining the project performance requirements for the completed structure.
- Selecting the tamper mass (weight) and drop height to correspond to the required depth of improvement.
- Estimating the degree of improvement that will result from dynamic compaction.
- Determining the applied energy to be used over the project site to produce the improvement.

19.6.2.1 Performance Requirements

Dynamic compaction densifies in-situ soils and thus improves the shear strength and reduces the compressibility of the in-situ soils. A baseline of in-situ properties should be established prior to commencing ground improvement using SPT or CPT methods. The approximate required level of improvement should be determined for the specific baseline testing procedure. Verification testing shall be conducted during the dynamic compaction operations to determine if the required amount of densification is being achieved.

19.6.2.2 Depth of Improvement

The depth of improvement is based on a number of variables including weight (mass) of the tamper, drop height, soil type, and average applied energy. The maximum depth of improvement is determined from the following equation.

$$D_{max} = n\sqrt{WH} \qquad \text{Equation 19-13}$$

Where,

D_{max} = Maximum depth of improvement (meters) (1 m = 0.3048 ft)

n = Empirical coefficient ranging from 0.3 to 0.8, but normally used as 0.5 for most soils and 0.4 is used for landfills

W = Mass of tamper (metric tonnes) (1 metric tonne = 2,205 pounds)

H = Drop height (meters)

The depth of improvement is also affected by the presence of soft or hard layers. Both types of layers absorb the energy imparted by the tamper and can therefore reduce the depth of improvement.

19.6.2.3 Degree of Improvement

As indicated above, the degree of improvement is typically measured using either SPT or CPT measurements. SPT or CPT tests are performed prior to and after dynamic compaction to monitor the amount of improvement imparted on the soil. Figure 19-20 provides a general indication of the amount of improvement from dynamic compaction.

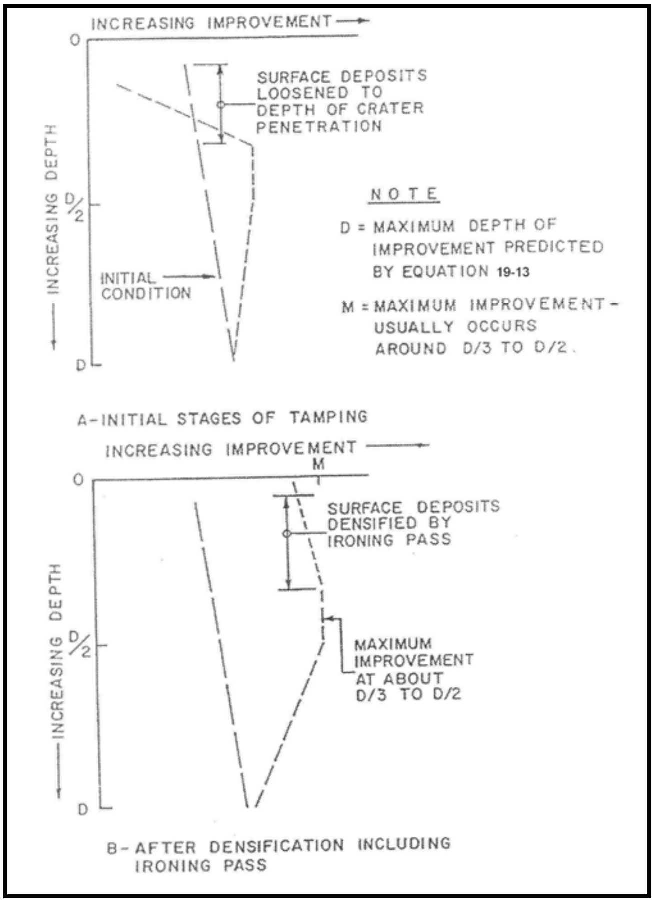

Figure 19-20, Dynamic Compaction Improvements vs. Depth (Ground Improvement Methods – August 2006)

The degree of improvement achieved is primarily a function of the average energy applied at the ground surface. Generally, the greater the amount of energy, the greater the degree of improvement; however, there are limitations to the maximum SPT or CPT values that can be achieved. These maximum values are listed in Table 19-12. These maximum values occur at improvement depth ranges of D/3 to D/2, above or below this range the test values would be less. These maximum values should only be used as a guide. The actual degree of improvement should be determined during and after the completion of dynamic compaction. The degree of improvement can continue to increase for months or, in some cases, years following the complete dissipation of excess pore pressures.

Table 19-12, Upper Bound Test Values after Dynamic Compaction (Ground Improvement Methods – August 2006)

Soil Type	Maximum Test Values	
	N-values (bpf)	Cone Tip Resistance (tsf)
Sand & Gravel	30 – 50	200 – 300
Sandy Silts	25 – 35	135 – 175
Silts & Clayey Silts	20 – 35	105 - 135
Clay fill & Mine spoil	20 – 40[1]	N/A
Landfills	15 – 40[1]	N/A

[1]Higher test values may occur because of large particles in the soil mass.

19.6.2.4 Energy Requirements

According to Ground Improvement Methods, "Dynamic compaction is generally undertaken in a grid pattern throughout the area. For this reason, it is convenient to express the applied energy in terms of average values. This average applied energy can be calculated on the basis of the following formula:"

$$\boldsymbol{AE} = \frac{(\boldsymbol{W})(\boldsymbol{H})(\boldsymbol{N})(\boldsymbol{P})}{(\boldsymbol{G})^2} \qquad \text{Equation 19-14}$$

Where,

AE = Applied energy
N = Number of drops at each specific drop point location
W = Tamper weight
H = Drop height
P = Number of passes
G = Grid spacing

The average applied energy is the sum of all different size tampers and drop heights. Normally, high energy is achieved using a heavy tamper dropped from a high height. This is frequently followed by the ironing pass (low level energy). The ironing pass is conducted using smaller sized tampers being dropped from lower heights. For planning purposes, the estimated required energy can be obtained from Table 19-13.

Table 19-13, Applied Energy Guidelines (Ground Improvement Methods – August 2006)

Soil Deposit	Unit Applied Energy ($ft\text{-}lb/ft^2$)	Percent Standard Proctor Energy[1]
Zone 1 Soils[2]	4,130 – 5,170	33 - 41
Zones 2 and 3[2]	5,170 – 7,230	41 - 60
Landfills	12,400 – 22,700	100 - 180

[1]Standard Proctor energy equals 12,400 $ft\text{-}lb/ft^2$

[2]Refer to Figure19-16

19.7 DEEP SOIL MIXING

Deep soil mixing is a ground improvement technique that mixes reagents into the soil at a specific depth to improve the in-situ soil properties without requiring excavation or removal. Deep soil mixing mixes the soil and reagent together, whereas grouting injects cementitious materials into the in-situ soil matrix to improve the soil. Grouting is discussed below. Deep soil mixing can be used for a variety of applications including excavation support, soil stabilization, settlement reduction, foundation support, and mitigation of liquefaction potential. Deep soil mixing is performed under many proprietary names, acronyms and processes worldwide. However, the basic concepts and procedures are similar for all techniques. The mixed soil product and the objectives of the mixing program can be divided into standard generic terms as presented in the table below:

Table 19-14, Deep Soil Mixing Generic Terms
(Ground Improvement Methods – August 2006)

Method of reagent injection	Wet (**W**) or Dry (**D**)
Method of reagent mixing	Rotary energy (**R**) or High-pressure jet (**J**)
Location of mixing action	End of drilling tool (**E**) or Along shaft (**S**)

These generic terms can be combined into four distinct processes of deep soil mixing (see Figure 19-21), WRS, WRE, WJE and DRE. Some of the possible combinations of deep soil mixing methods do not exist. For example DJE (dry, jet end) does not exist. Jetting is a wet method and, therefore, could not be used with a dry mix application.

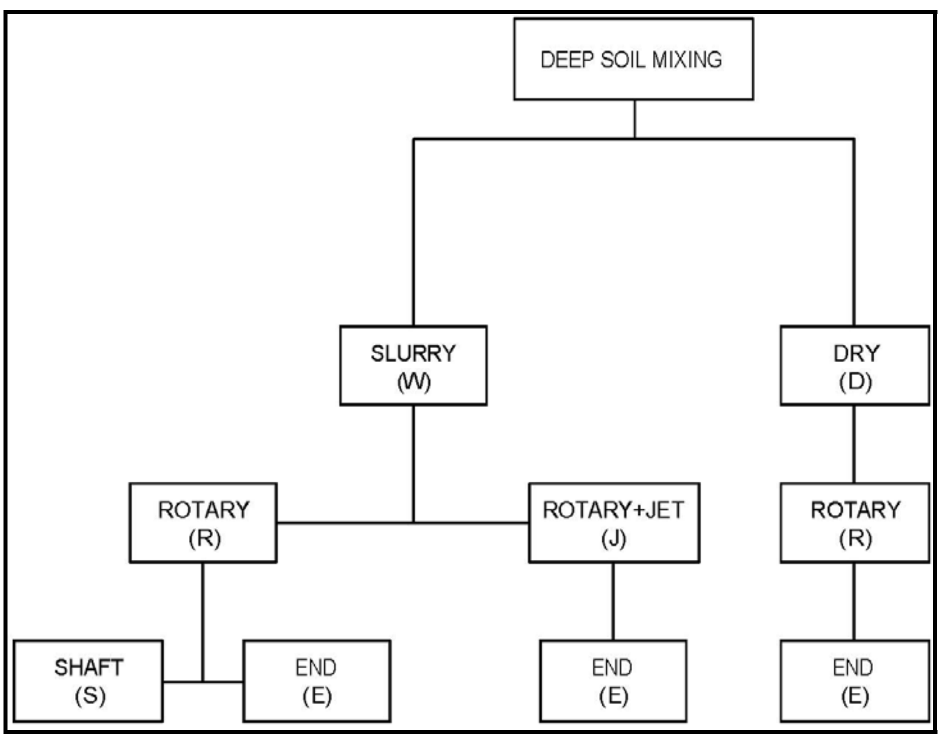

Figure 19-21, Generic Classification of Deep Soil Mixing Techniques
(Ground Improvement Methods – August 2006)

The four processes discussed previously can be divided into two groups as indicated in Table 19-15.

Table 19-15, Deep Soil Mixing Groups
(Ground Improvement Methods – August 2006)

Wet Deep Soil Mixing Methods	WRS, WRE, WJS	Refers to wet, single or multi-auger, block or wall developed for large-scale foundation improvement in any soil. Primary reagents are cement-based.
Dry Deep Mixing Methods	DRE	Refers to dry, single-auger column technique developed for soil stabilization and reinforcement of cohesive soils. Primary reagents are granular or powdered lime for lime columns and cement or lime-cement mixtures.

19.7.1 Analysis

Wet, deep soil mixing methods are typically used for large-scale structural support improvement, while dry deep soil mixing methods are used primarily for soil stabilization/reinforcement and settlement reduction. Discussed in the following paragraphs are applications, of wet and dry deep soil mixing that are typical for transportation related projects. For other applications see Ground Improvement Methods.

Wet deep soil mixing methods have been used to stabilize soil to provide an improved foundation bearing capacity and for seismic stabilization. The most common usage is for settlement control and/or shear strength improvement under embankments. Under this usage wet deep soil mixed columns are constructed in grid or lattice geometry to provide additional resistance to bending. This same method can be used to improve the mass shear strength of a potentially liquefiable soil as well as contain liquefaction propagation.

Dry deep soil mixing methods such as lime, cement, or lime-cement columns have been used to improve soft, cohesive soils. Lime-cement columns have been used to reduce total and differential settlements using rationale similar to stone columns. These columns are stiffer and relatively less compressible than the surrounding soil; therefore, carry a greater portion of the applied load thus reducing total and differential settlement. The amount of settlement reduction is a function of the area replacement ratio and the stress concentration ratio, which is a function of the column stiffness compared to the untreated soil. These types of columns are used to reinforce existing soils by increasing the mass shear strength, thus increasing the stability of embankments and slopes. Typically, the columns are placed in a grid pattern under the embankments and in interconnected rows under the slope to provide sufficient resistance to bending. Lime, cement, or lime-cement columns can be used to increase the stability of anchored sheet pile walls. The columns increase the passive earth pressure at the toe of the wall. In addition, columns placed behind the wall can reduce the lateral earth pressure acting on the sheet piles.

19.7.2 Advantages and Disadvantages/Limitations

19.7.2.1 Wet Deep Soil Mixing Methods

The advantages of wet deep soil mixing are, it can be performed to depths up to 100 feet and can, conceptually, be used for most subsurface conditions, from soft, plastic clays to medium dense sands and gravels with cobbles. However, this method is primarily used to improve soft cohesive and loose to medium dense cohesionless soils. Deep soil mixing uses the in-situ soil, making this method more economical than removal and replacement. The problems associated with disposal of the waste material are considerably reduced in an amount proportional to the percentage of additives used and the moisture content of the in-situ soils. The construction is a drilling process which is ideal in noise and vibration sensitive areas.

The disadvantages/limitations of wet deep soil mixing are the relative high cost of mobilization of the mixing equipment plus the cost of accompanying auxiliary batch plants. Wet deep soil mixing is uneconomical for small projects. A more extensive geotechnical exploration is required prior to using wet deep soil mixing than is typical. In addition, bench scale testing must be conducted and may require several months to complete. Dense cohesionless soils can not be readily penetrated by the existing deep soil mixing equipment. The amount of spoil produced by deep soil mixing is generally less than for some ground improvement methods. Spoil generation can range from thirty to one-hundred percent depending on project specifics, equipment and methods used, and in-situ moisture content. Disposal of this spoil can add significant cost to a project. There is a lack of well developed design and analysis models available. Lastly, there is no standardized method of quality control testing, making design verification difficult and subjective.

19.7.2.2 Dry Deep Soil Mixing Methods

One advantage of dry deep soil mixing methods in soft clay is that it often provides an economic benefit when compared to other conventional foundation methods. This advantage is based on several project factors including size, weight, and flexibility of the structure, depth, and shear strength of the compressible layer, the risks, and consequences of failure and the effects of lowering the groundwater table. Using lime or lime-cement columns can reduce the consolidation time required beneath a roadway embankment by increasing the permeability or stiffness of the columns. Another advantage to the dry deep soil mixing method is little to no spoil is generated by this method, thus eliminating the high cost of spoil disposal.

One of the disadvantages/limitations of dry deep soil mixing methods is the full strength of the columns may not be mobilized when the pH of the groundwater is acidic or the content of carbon dioxide (CO_2) is high. Low strength development should also be anticipated when mixing non-reactive cohesive soils (clays lacking pozzolans). The air-driven injection process may accumulate large quantities of air in the ground potentially causing heave of the adjacent ground surface. This problem can be eliminated by adding mixing paddles to the mixing tool and/or substantially increasing the mixing time. The creep strength of the columns and the shear strength of the stabilized soil is time dependent. Therefore, several months may be required to perform the laboratory bench scale testing. The average shear strength of the stabilized soil has to be at least three to five times the initial shear strength before dry deep soil mixing becomes economical. There is a lack of well developed design and analysis models available. Lastly, there is no standardized method of quality control testing, making design verification difficult and subjective.

19.7.3 <u>Feasibility</u>

The feasibility of using deep soil mixing shall be determined prior to recommending this ground improvement method. The feasibility evaluation includes, but is not limited to, a site investigation, a feasibility assessment, and preliminary testing (bench scale testing).

19.7.3.1 Site Investigation

The site investigation required for deep soil mixing exceeds the requirements contained in this Manual. If deep soil mixing is selected as an alternate ground improvement method, then, additional site specific information will be required. The proposed site investigation plan shall be submitted to the GDS and the PCS/GDS for concurrence prior to execution. Prior to commencing the site investigation, observations of the proposed construction area should be made to include ground surface condition, the presence of overhead or underground utilities, site access, and any other observations that could affect the ability to use this method. It should be noted that typically the equipment used for deep soil mixing is relatively large and will require more space to operate in. In addition, use of the wet methods may generate large amounts of spoil, and it should be determined if there is adequate space on site to store this material. The site investigation should include the following items:

- Evaluation of the subsurface: predominant soil type; existence of any obstructions; existence and percentage of organic matter
- Natural moisture content
- Engineering properties: strength and compressibility
- Classification properties: moisture-plasticity relationship and grain-size distribution
- Chemical and mineralogical properties to include assessment for the presence of pozzolanic materials, including soluble silica and alumina, which can affect lime reactivity only
- Ground water levels

19.7.3.2 Assessment

Deep soil mixing is best used when the subsurface conditions are soft to loose with no obstructions to depths no greater than 100 feet. There should be unrestricted overhead clearance and a need for relatively vibration free ground improvement methods. Deep soil mixing will cause the temporary loss of in-situ soil strength, which may affect adjacent structures. The assessment should review the information obtained from the site investigation. Selected soil chemical properties are provided in the table below.

Table 19-16, Favorable Soil-Chemistry Factors (Ground Improvement Methods – August 2006)

Property	Favorable Soil Chemistry
pH	> 5
Natural moisture content	< 200 (dry method) < 60 (wet method)
Organic content	< 65 (wet method)
Loss on Ignition	< 10
Humus Content	< 1
Electrical conductivity	0.4 mΩ/cm

19.7.3.3 Preliminary Testing

After assessing the viability of soil for deep soil mixing, samples should be prepared to determine the water, soil, reagent ratios as well as determining the time required for mixing.

The samples should then be tested for unconfined compressive strength at various curing times to determine strength gains with time. This entire process can be called preliminary or bench scale testing. The preliminary testing results will assist in narrowing the potential improvements levels that can be achieved in the field. These results should be compared to the typical results presented in the table below. It is important to note that very important variables associated with equipment mixing capabilities, such as rate of penetration and withdrawal, mixing energy, and vertical circulation of materials, cannot be modeled by the laboratory testing program.

Table 19-17, Typical Improved Engineering Properties (Ground Improvement Methods – August 2006)

Property	Typical Range
Unconfined Compressive Strength, q_u	Cohesionless Soils – 29 – 725 psi Cohesive Soils – 29 – 435 psi
Hydraulic Conductivity, k	10^{-4} – 10^{-7} cm/s
Young's Modulus (E_{50}) [Secant Modulus at 50% q_u]	100 – 300 q_u
Tensile Strength (wet mix)	8 – 14 percent of q_u
Poisson's Ratio	0.19 – 0.45 typically 0.26

Provided in the table below are guidelines related to the penetration, mixing speed, water cement ratio, and reagent content typically used in practice.

Table 19-18, Mixing Guidelines (Ground Improvement Methods – August 2006)

Reagent Content	9-1/2 – 22-1/2 pcf
Mixing Rotational Speed	20 – 45 rpm
Penetration Rate	1 yd/min
Water Cement Ratio	0.6 – 1.3 but 1.0 is normal

According to Ground Improvement Methods, "Much recent research and interest has been directed toward developing indicators of the potential efficiency of the mixing process that would produce a more homogeneous in-situ product of higher strength. It has been suggested by Japanese researchers that efficiency of a particular system can be established or expressed in terms of 'the number of mixing per *yard*, T,' which is related to certain operational and reagent injection characteristics as follows:"

$$T = N\left[\left(\frac{R_p}{S_p} \bullet \frac{W_i}{W}\right) + \frac{R_w}{S_w}\right] \qquad \text{Equation 19-15}$$

Where,

N = Total number of mixing blades
S_p, S_w = Penetration and Withdrawal speed (yard/min)
R_P, R_w = Blade rotation speed during penetration and withdrawal (rpm)
W_i = Stabilizer (reagent) injection on penetration (pcf)
W = Total amount of stabilizer (reagent) (pcf)

T should be greater than 350 for clays and range from 400 to 450 for peaty soils according to the research to develop a good quality product.

19.7.4 Design

Deep soil mixed columns are designed similarly to stone columns in that unit cell concepts, stress ratios (n) and area replacement ratios (α_s) are used for design. For settlement reduction, area replacement ratios on the order of 0.2 to 0.3 are used for triangular or square column patterns. Determining the strength to support the embankment load, maximizing the benefit of arching between columns, and providing the required global shear strength to ensure stability develops the optimum design spacing. Larger area replacement ratios are indicative of more stringent settlement criteria. Large area deep soil mixing columns can be used to support structures provided stability (bearing, sliding and overturning) and performance (total and differential settlement) are satisfied. Deep soil mixing columns can also be used to mitigate the potential for liquefaction by either confining the materials that will liquefy or by increasing the Cyclic Resistance Ratio (CRR) (see Chapter 13) through increasing the shear strength of the soil.

The area replacement ratio (α_s) is defined as:

$$\alpha_s = \frac{A_s}{A} \qquad \text{Equation 19-16}$$

Where,

α_s = Area replacement ratio
A_s = Area of the soil mixed column
A = Total area within the unit cell

The transfer of the applied load to the soil mixed columns from the in-situ soils depends on the relative stiffness of the soil mixed columns to the in-situ soils as well as the spacing and diameter of the soil mixed columns. Because the soil mixed columns and the in-situ soils deflect (strain) approximately equally, the soil mixed columns must be carrying a greater portion of the load (stress) than the in-situ soils. This concept has also been called the equal strain assumption. This concept has been proven by both field measurements as well as finite element analysis. The relationship between the stress in the stone column and the stress in the in-situ soil is defined in the following equation:

$$n = \frac{\sigma_s}{\sigma_c} \qquad \text{Equation 19-17}$$

Where,

n = Stress ratio or stress concentration
σ_s = Stress in the soil mixed column
σ_c = Stress in the surrounding soil

Equilibrium of vertical forces for a given α_s is provided by the following equation.

$$q = \sigma_s \alpha_s + \sigma_c (1 - \alpha_s)$$ Equation 19-18

Where,

q = Average stress on the unit cell

The stresses in the soil mixed column and the surrounding soil in the unit cell can be determined by rearranging the above equation.

$$\sigma_c = \frac{q}{[1 + (n-1)\alpha_s]}$$ Equation 19-19

$$\sigma_s = \frac{nq}{[1 + (n-1)\alpha_s]}$$ Equation 19-20

According to Ground Improvement Methods, "The total undrained shear resistance τ of the stabilized soil is assumed to correspond to the sum of the shear strengths of the column and the soil between the columns and can be evaluated from"

$$\tau = \tau_f \alpha_s + C_u (1 - \alpha_s)$$ Equation 19-21

Where,

τ_f = Undrained shear strength of soil mixed column

C_u = Undrained shear strength of soil between columns

α_s = Area replacement ratio

"Typical area *replacement* ratios are on the order of 0.20 to 0.40 and are varied until the targeted minimum total undrained shear resistance of the stabilized soil is calculated. It is anticipated that area replacement ratios of 0.20 to 0.33, and stress ratios of between 4 and 6, would *be* used typically for either block- or column-type patterns *(see Figure 19-22)*. The reduction in settlement is attributed to the concept that *the soil mixed* columns that are stiffer than the adjoining soil will carry more load."

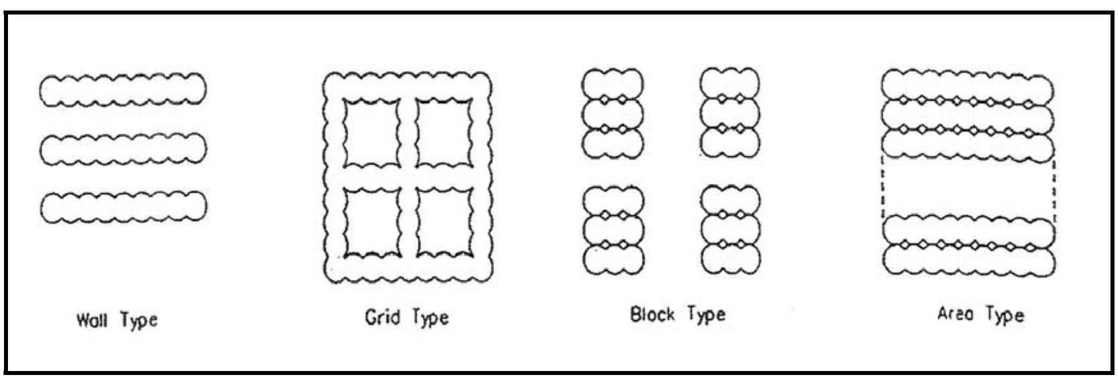

Figure 19-22, Deep Soil Mixing Treatment Patterns
(Ground Improvement Methods – August 2006)

19.7.5 Wet Soil Mix Material Properties

The properties of wet soil mixing are influenced by the soil type and chemistry, in-situ water content, amount of reagent used, water-reagent ratio of slurry, degree of mixing, curing environment, construction process and equipment, spoil generated, and age. The in-situ strength of the treated soil can be one-half to one-fifth of the strength measured in the laboratory. Therefore, the strength of the mixed soil prepared in the laboratory should only be used as an indication of the level of improvement that is achievable in the field. Typically, the longer a mixed soil cures, the greater the increase in strength. Field testing has shown increases in mixed soil strength up to 6 months after mixing. Provided in the following table are typical compressive shear strengths.

Table 19-19, Typical Improved Compressive Strength, Wet Mix Method (Ground Improvement Methods – August 2006)

In-Situ Soil	Improved Compressive Strength (psi)
Organic and very plastic clays	175
Soft clays	60 – 220
Medium/hard clays	100 – 360
Silts	145 - 435
Fine to medium sands	220 – 725

19.7.6 Dry Soil Mix Material Properties

Dry mix methods should be used when the in-situ soils consist of soft clays with in-situ moisture contents between sixty and one-hundred and twenty percent and where the required increase in strength is less than 145 psi. Typically, dry mix methods use either plain lime or lime-cement as the reagent. The lime-cement modified soils will have higher shear strength than the lime only stabilized soils. As with wet mixed soils, the shear strength obtained from laboratory prepared specimens should be reduced. The shear strength should be reduced by approximately one-third to one-half, but should be no greater than 8.4 ksf.

19.7.7 Verification

The properties of the improved ground require verification to ascertain whether the requirements of the project are being met. The contractor should be required to conduct laboratory (bench scale) testing to verify that proposed construction methods and mixes will achieve the requirements of the contract. After completion of the mixing, either in-situ testing or obtaining cores for laboratory testing should be performed. The in-situ testing can consist of cone penetrometer testing (CPT), dilatometer testing (DMT), standard penetration testing (SPT), or pressuremeter testing (PMT).

19.8 GROUTING

According to Ground Improvement Methods, “Grouting comprises a variety of techniques that employ injection of a range of materials into soil or rock formations, via boreholes, to alter the physical characteristics of the formation when the materials set. More specifically, grouting can be used to fill fissures and voids in rock, to fill voids between the ground and overlying structures, and to treat soils to enhance strength, density, permeability, and/or homogeneity." The type of grouting used is based on the anticipated/required results and the soil/rock that the grouting is being used in. A successful grouting program consists of a detailed geotechnical investigation, active monitoring during construction, and verification that the grouting program is meeting the project requirements.

The geotechnical investigation is more detailed than is normally performed to identify in-situ conditions that could affect the effectiveness of the grouting program. The results of this detailed investigation are used to select the type of grouting, as well as the grouting materials. In addition, the investigation will aide in determining the potential effectiveness of the grouting program. To improve effectiveness, a real time monitoring plan is required, which allows for field adjustments to the grouting program to account for changes in subsurface conditions. Finally, a comprehensive grouting program shall include a means of verifying that the required results are being achieved.

The definitions contained in the Ground Improvement Methods manual are used in this Manual. The Ground Improvement Methods manual identifies two principle types of grouting which are listed in the table below. Figure 19-23 provides schematics of the various types of grouting.

Table 19-20, Types of Grouting Method
(Ground Improvement Methods – August 2006)

Principle Type of Grouting	Specific Type of Grouting
Rock Grouting	Fissures (using High Mobility Grouts (HMG))
	Voids (natural and artificial, using Low Mobility Grouts (LMG))
Soil Grouting	Permeation (using HMG and solution grouts)
	Compaction (or displacement)
	Jet (or replacement)
	Fracture (including compensation grouting)

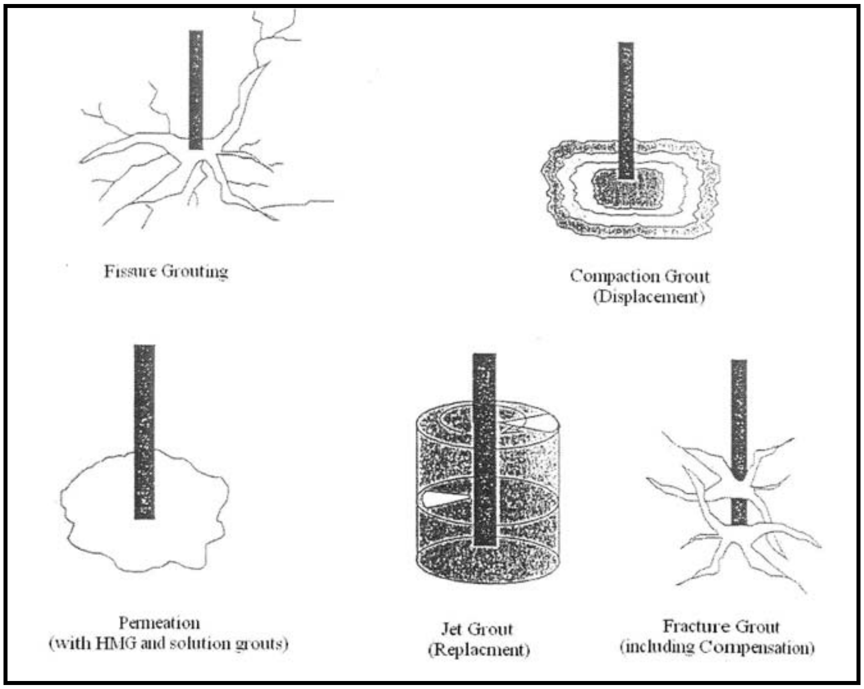

Figure 19-23, Types of Grouting Schematic
(Ground Improvement Methods – August 2006)

19.8.1 Grout Materials

There are four categories of grouting materials, which are listed below:

1. Particulate (suspension or cementitious) grout
2. Collodial solutions
3. Pure solutions
4. Miscellaneous materials

Category 1 grouts are comprised of mixtures of water and particulate solids. The particulate solids may consist of cement, fly ash, clays or sands. These mixtures are stable and have cohesion and plastic viscosity increasing with time. Due to their basic characteristics and relative economy, these grouts remain the most commonly used for both routine waterproofing and ground strengthening. The water to solids ratio is the prime determinant of their properties and basic characteristics such as stability, fluidity, viscosity, and strength durability. Neat cement or clay/bentonite-cement grouts are comprised of Portland cement or microfine cement depending on the size requirements of the grout. Figure 19-24 shows the increase in apparent viscosity with time for these grouts and Figure 19-25 shows grain-size distribution of various cements.

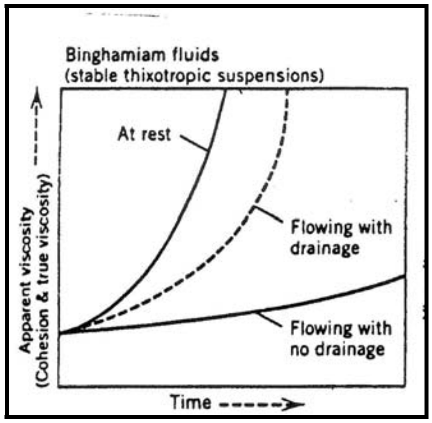

Figure 19-24, Viscosity versus Time for Category 1 Grouts (Ground Improvement Methods – August 2006)

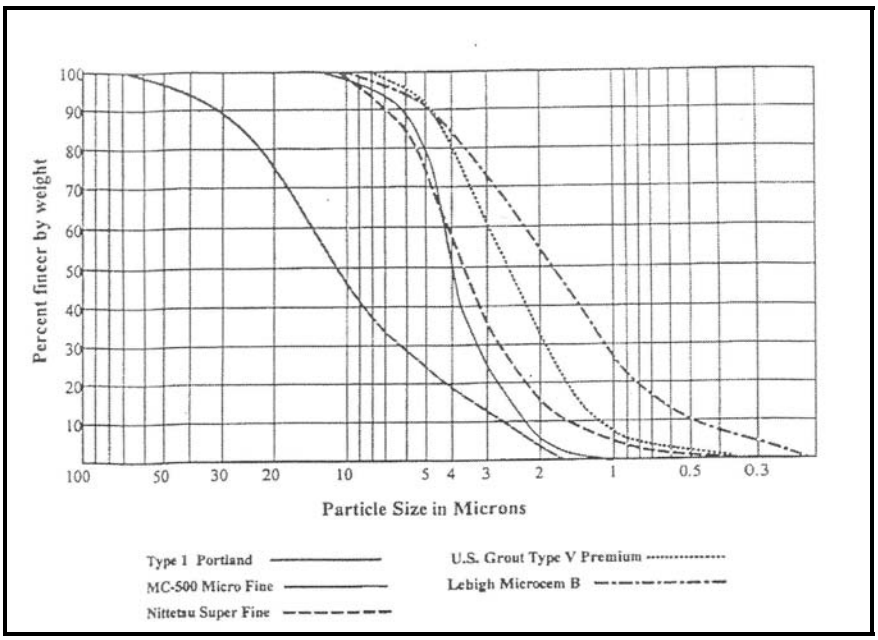

Figure 19-25, Grain-Size Distribution of Cements (Ground Improvement Methods – August 2006)

Category 2 and 3 grouts, commonly called solution or chemical grouts, are typically subdivided based on component chemistries; for example, silicate based (Category 2) (colloidal) or resin based (Category 3) (pure solution). Figure 19-26 provides an indication of the change of viscosity with time for these grouts. Category 2 grouts are colloidal solutions that are comprised of mixtures of sodium silicate and a reagent, which when mixed, change viscosity over time to a gel. Sodium silicate is an alkaline, colloidal aqueous solution, while the reagents may be organic or inorganic (mineral). The common types of organic reagents are monoesters, diesters, triesters and aldehydes. These reagents react with the sodium silicate to produce acid as a by-product and can produce either a soft or hard gel depending on the concentration of each compound. The inorganic reagents contain cations that are capable of neutralizing the silicate alkalinity. Typical inorganic reagents are sodium bicarbonate and sodium aluminate. The relative proportions of silicate and reagent will be determined by their own chemistry and concentration, the desired short- and long-term properties, such as gel setting time, viscosity, strength, syneresis and durability, as well as cost and environment acceptability.

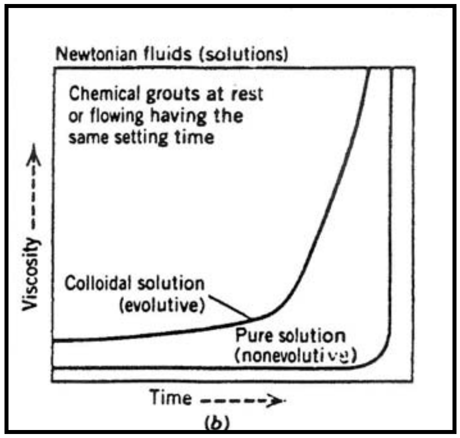

Figure 19-26, Viscosity versus Time for Category 2 and 3 Grouts (Ground Improvement Methods – August 2006)

Category 3 grouts are known as pure solutions since these grouts consist of resins. The resins are solutions of organic products in water or a nonaqueous solvent that are capable of causing the formation of a gel with specific mechanical properties under normal temperature conditions and in a closed environment. These grouts exist in the following forms, characterized by the mode of reaction or hardening:

- Polymerization – Activated by the addition of a catalyzing agent (polyacrylamide resins)
- Polymerization and Polycondensation – Arising from the combination of two components (epoxies or aminoplasts)

The setting times for these grouts is adjusted by varying the proportions of the reagents or components. According to Ground Improvement Methods, "Resins are used when particulate grouts or colloidal solutions prove inadequate, for example when the following grout properties are needed:

- Particularly low viscosity
- Very fast gain in strength (a few hours)
- Variable setting time (few seconds to several hours)
- Superior chemical resistance
- Special rheological (psuedoplastic)
- Resistance to high groundwater flows"

In applications where the durability of the grout is important, resins are typically used for both strength and waterproofing. Resins may be divided into four subcategories as indicated in Table 19-21.

Table 19-21, Types, Use, and Applications of Resins
(Ground Improvement Methods – August 2006)

Type of Resin	Applicable Ground Type	Use/Application
Acrylic	Granular, very fine soils Finely fissured rock	Waterproofing by mass treatment Gas tightening (mines, storage) Strengthening up to ~15 tsf Strengthening of a granular medium subjected to vibrations
Phenol	Granular, very fine soils	Strengthening
Aminoplastic	Schists and coals	Strengthening (by adherence to materials of organic origin)
Polyurethane	Large Voids	Formation of a foam that forms a barrier against running water (using water-reactive resins) Stabilization or localized filling (using two-component resins)

There are only two types of polyurethanes that are appropriate for grouting. These types are listed in Table 19-22.

Table 19-22, Polyurethane Types
(Ground Improvement Methods – August 2006)

Polyurethane Type	Properties
Water Reactive	Liquid resin reacts with groundwater to form either flexible (elastomeric) or rigid foam These resins take two forms: • Hydrophobic – react with water, but repel it after the final (cured) product has formed • Hydrophilic – react with water, but continue to physically absorb it after the chemical reaction has been completed
Two Component	Two compounds in liquid form react to provide either a rigid foam or an elastic

Category 4 grouts (Miscellaneous grouts) are composed of organic compounds or resins. These grouts are used primarily for strengthening and waterproofing, but may also have very specific qualities such as resistance to erosion or corrosion, and flexibility. The use of Category 4 grouts may be limited by specific concerns such as toxicity, injection, handling difficulties, and cost. In addition, many of these grouts are proprietary in nature, which can make their use difficult at best. Category 4 grouts are composed of hot melts, latex, polyesters, epoxies, furanic resins, silicones, and silacols. Some of these types have limited use in ground improvement. Category 4 grouts should only be used if there are either no other options or if the grouting system (grout and application of the grout) is fully understood by both the designer and the contractor.

19.8.2 Rock Grouting

There are two types of rock grouting: rock fissure grouting and void filling. Both types of grouting are discussed briefly in the following sections.

19.8.2.1 Rock Fissure Grouting

The grouting of rock fissures is primarily used to provide hydraulic cut-offs and has the added benefit of binding the rock mass together thus improving the load bearing capability. Rock fissure grouting typically has limited applications on transportation projects. However, rock fissure grouting can be used to stabilize rock slopes, remediate road tunnels, repair drilled shafts, and seal of drilled shaft boreholes from the in-flow of ground water. The variability of the rock mass can make this ground improvement technique extremely difficult to predict the results of using the method. Because of the variability in the rock mass, often a design phase test program is conducted to determine the effectiveness of the rock fissure grouting program. Using the results of the test program, the final design can be completed and a program cost can be estimated.

The use of rock fissure grouting has the advantage of being less expensive when compared to other repair options of weak rock, such as removal, replacement, or abandoning the site. However, the actual cost of rock fissure grouting can vary considerably because of potential variation of the rock mass within the site boundaries. Further, poor field practices can lead to unsatisfactory performance of the rock grouting. These include inducing uplift that results from excessive pressures, premature plugging of fissures, unsuitable injection methods or formulations or by inappropriate drilling and flushing methods and improper hole spacing or improper orientation of the grout holes.

The primary purpose of this form of rock grouting is the sealing of cracks and fissures within the rock mass. The main consideration in rock grouting is the grain-size of the particulate grout compared to the width of the rock fracture to be grouted.

$$N_R = \frac{f_w}{D_{95}} \qquad \text{Equation 19-22}$$

Where,

N_R = Groutability ratio of rock
f_w = Fissure width
D_{95} = Grout diameter at 95 percent finer
$N_R > 5$ – Grouting consistently possible
$N_R < 2$ – Grouting not possible

While the fissure width cannot be changed, the fineness of the grout can be controlled, thus producing a groutability ratio that can be increased to greater than two. Rock grouting with particulate materials normally falls into one of the categories indicated in Table 19-23.

Table 19-23, Rock Grouting Categories
(Ground Improvement Methods – August 2006)

Rock Grouting Category	Description
Curtain	Drilling and grouting of two or more lines of grout holes to an impermeable material to produce a barrier to seepage.
Area	Grouting a shallow zone in a particular area by utilizing grout holes arranged in a pattern or grid to mechanically improve fractured or jointed rock.
Tunnel	Used to fill voids behind tunnel liners, treatment of material surrounding the bore or seepage control. Pre-excavation grouting from the surface or the face may be required for ground strengthening and water control.
Backfilling	Filling subsurface exploration boreholes and grout holes is important to maximize structural stability, to control water, or to prevent passage of contaminants to underlying strata.

19.8.2.2 Rock Void Grouting

Rock void grouting is used to fill natural (karstic limestone features or salt solution cavities) voids or man-made (mining activities) voids. Typically, neither of these features occurs in South Carolina. However, there are some localized areas of karstic limestone features caused by localized dewatering for mining activities. Ground Improvement Methods includes slabjacking (mudjacking) as a subset of rock void grouting. Slabjacking is the process of injecting grout under pressure to raise and relevel concrete paving (typically bridge approach slabs) that have settled (see Figure 19-27).

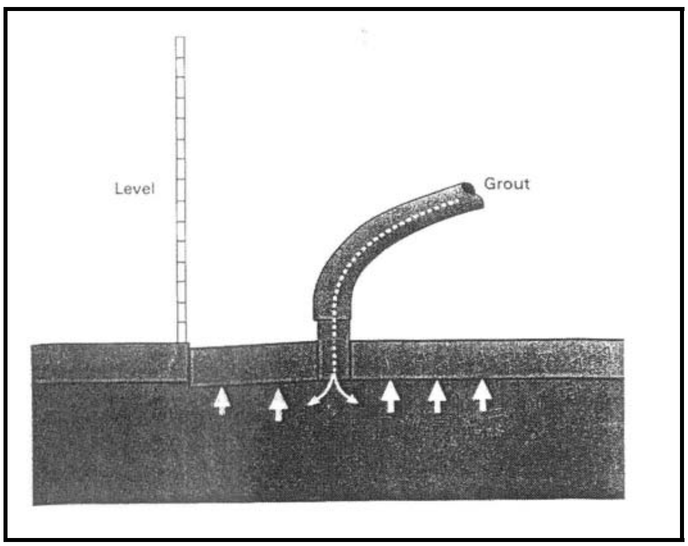

Figure 19-27, Slabjacking Schematic
(Ground Improvement Methods – August 2006)

Rock void grouting can also be used for the remediation of some scour issues. However, it will not be discussed in this Manual. Contact the PCS/GDS for guidance in the use of this method for remediation of scour. As indicated previously, slabjacking is used to correct the settlement of concrete slabs placed over compressible soils or to replace soils that have eroded away from beneath the slab. Typically, this method is used to correct problems associated with the vertical displacement of bridge approach slabs. According to Ground Improvement Methods, "Slabjacking procedures include raising or leveling, under-slab void filling (no raising), grouting slab joints, and asphalt subsealing. Most slabjacking uses a suite of cementitious grouts, incorporating bentonite, sand, ash and/or other fillers, as dictated by local preference and the project conditions and goals. Certain proprietary methods use expanding chemical foams to create uplift pressures. Best results (when no cracking is caused to the slabs) are obtained when the slabjacking is uniformly and gradually conducted. Slabjacking can also be used to "pump" *sections of rigid pavements* that have sunk below the adjoining section so that the expansion joint may be repaired are have it's functionality restored."

Slabjacking has the following advantages:

- Frequently, the most economical repair method
- Usually faster than other solutions, especially compared to removal and replacement
- Planned so that there is little disruption to the existing facility, and can be performed at times of light or no traffic
- The equipment needed to perform the slabjacking operation can be removed from the repair location, providing for maximum accessibility
- Increased load capacity of the slab is provided
- The useful life of the concrete pavement is extended
- A smoother riding surface is established

Following are the disadvantages of slabjacking:

- Cracks already present may tend to open up when the slab is treated, unless great care is taken with the process
- Slabjacking may not be cost-effective on small projects
- The original cause of the settlement is not addressed

The feasibility of using slabjacking should be based on the cost of slabjacking versus the cost of removal and replacement of the slab. Included in this evaluation should be the time required for both operations and if a roadway must be closed to perform this operation. In addition, slabjacking should not be considered when the slab is severely cracked.

After determining that slabjacking is feasible, the design should begin with understanding the underlying problem and determining the desired results of the slabjacking. If the underlying problem is settlement of soft or organic soils, then, future slabjacking may be required. Regardless of the cause of the problem, the engineer should accurately specify the required performance and tolerances for the project. Another consideration is the appearance of the finished surface. Most slabs that have settled contain some cracks. The cracks will remain visible even if the slabjacking process does not create new cracks. Further, the restored slab will also contain patches from the injection holes. The injection holes are usually on 5- to 6-foot grid spacing. The objectives of slabjacking are to fill voids and raise the slab approximately to its original elevation, without causing additional damage to the slab. Instrumentation, as simple as a string line can provide this, although the use of lasers is more accurate.

19.8.3 Soil Grouting

Soil grouting programs are used to achieve a variety of ground improvement objectives. The two main objectives of a grouting program are, first, water control and waterproofing and second, structural improvement. Waterproofing is used mainly in conjunction with new construction and water control is used mainly in conjunction with remedial applications. Structural grouting is used to improve the density of a soil, raise settled structures, control settlement, underpin, mitigate liquefaction, and control water. There are four different types of grouting that can be used on soil:

1. Permeation
2. Compaction
3. Jet
4. Soil Fracture

All four of these types of grouting can be used for water control, waterproofing and structural enhancement. These four types are discussed in greater detail in the following sections. Soil grouting has a distinct economic advantage over removal and replacement. Grouting is also generally less disruptive to the surrounding work area. Soil grouting also has some disadvantages, such as compaction grouting in fine saturated soils. Instead of squeezing the pore water out, the soil may simply displace and not consolidate or densify. Permeation grouting using certain chemical grouts may represent toxicity dangers to the groundwater and underground environment. Low toxicity chemical grouts are now available and should be specified except for unusual circumstances. Each grouting method can cause ground movement and structural distress.

The general limitation of soil grouting is the soil type to be treated. Although the range of soil grouting available encompasses most soil types, individual methods are limited to specific soils as shown in Figure 19-28.

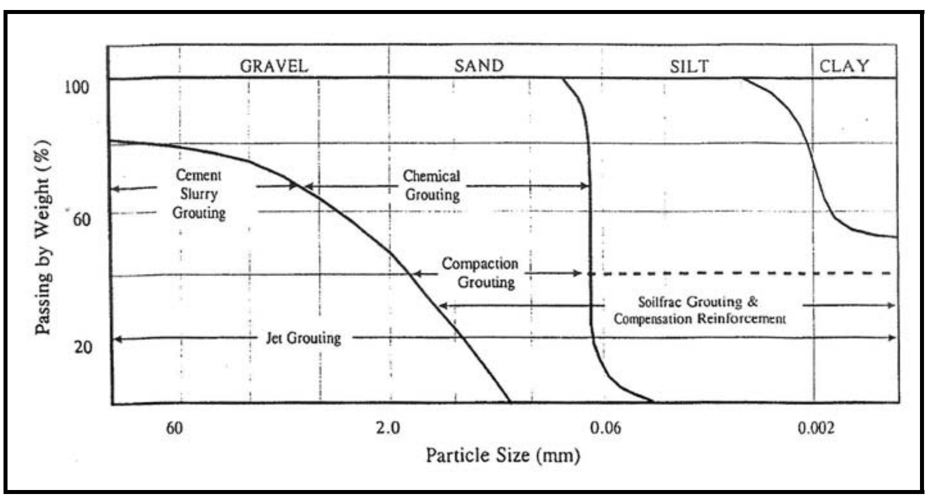

Figure 19-28, Range of Applicability of Soil Grouting Techniques (Ground Improvement Methods – August 2006)

Grouting is normally used to solve construction problems related to geological anomalies or environmental conditions. Soil grouting uses the existing soils, improving these soils, by grouting to correct deficiencies in the soil. According to Ground Improvement Methods, "Grouting of a soil involves the following sequential steps:

- Establishing specific objectives for the grouting program (designer)
- Defining the geometric and geotechnical project conditions (designer)
- Developing an appropriate grouting program design and compaction specifications and contract documents (designer)
- Planning the grouting equipment needs and procedural approach (contractor)
- Monitoring and evaluation of the grouting program (designer and contractor)"

The pregrouting subsurface exploration is more detailed than is normally required and should include continuous sample and laboratory tests. These tests should included grain-size analysis, density, permeability, pH, and other soil index properties.

The subsurface exploration should identify the extent that grouting can be utilized and areas or site conditions where grouting cannot be utilized. Subsurface stratigraphy can be well defined by continuous sampling. Small, fine-grained lenses should be noted, since these layers can retard the progression of some types of grouting. Considerably more descriptive detail is required on the boring log to be used by a grouting specialist than is typically shown on a standard boring log. Past uses of the site should be identified, such as the presence of abandoned wells, cisterns, cesspits, etc. These items can absorb the grout and either increase the grout take or cause no ground improvement. In addition, the presence of utilities should be noted, since the bedding materials of some utilities can cause a loss of grout as well. The grouting contractor should record every anomaly encountered in the drilling and grouting operations. These anomalies should be explained and evaluated prior to continuing drilling and grouting operations. Finally, the groundwater should be well understood. Samples of the groundwater should be tested for compatibility with the grouts to be used. Different levels of pH will determine which types of grout can be used at a site. In addition, grout specimens should be prepared in the laboratory using samples of groundwater to determine if there will be any interaction between the grout and the groundwater. Further, additional samples should also be prepared using water from the actual source. The direction and rate of groundwater flow should also be established during the subsurface investigation.

19.8.3.1 Permeation Grouting

Permeation grouting uses a variety of grout materials, particulate, colloidal and solution, to permeate the soils. The choice of which grout material to use is based on the grain-size distribution of the soil to be grouted (see Figure 19-29). Permeation grouting is an option in appropriate soils for the following applications:

- Waterproofing, typically for remedial purposes
- Settlement control
- Liquefaction retrofit mitigation by increasing density and displacing pore water

For permeation grouting to be successful, the soils must be "groutable". Groutability should be based on the permeability of the soil. A first estimate of permeability, and thus groutability, is based on the fines content (i.e., the percentage of material passing the #200 sieve). Table 19-

24 and Figure 19-30 provide the approximate percentage of material passing the #200 sieve and the groutability of a soil.

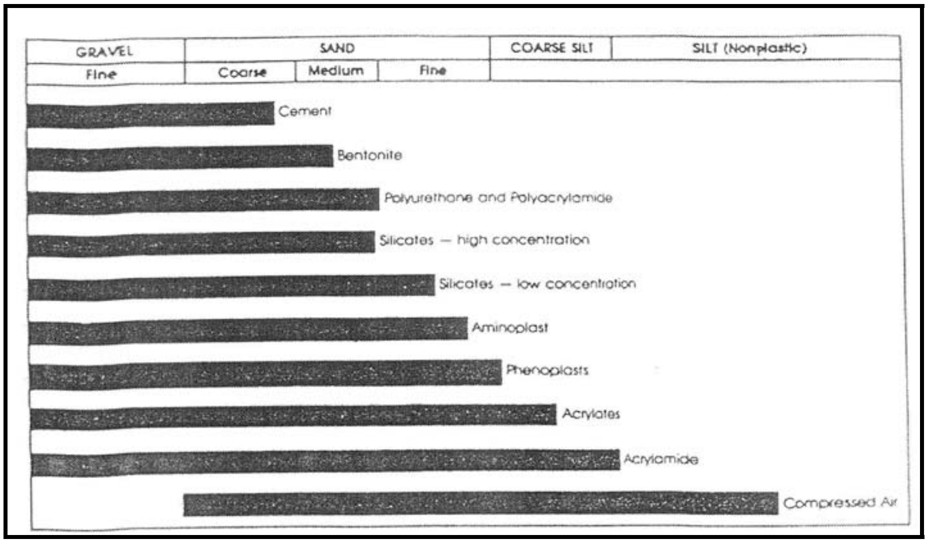

Figure 19-29, Penetrability of Various Grouts versus Soil Type (Ground Improvement Methods – August 2006)

Table 19-24, Groutability Guidelines

Percent Passing #200 Sieve	Description
<12	Readily groutable
12 – 15	Moderately groutable
15 - 20	Marginally groutable
> 20	Non-groutable

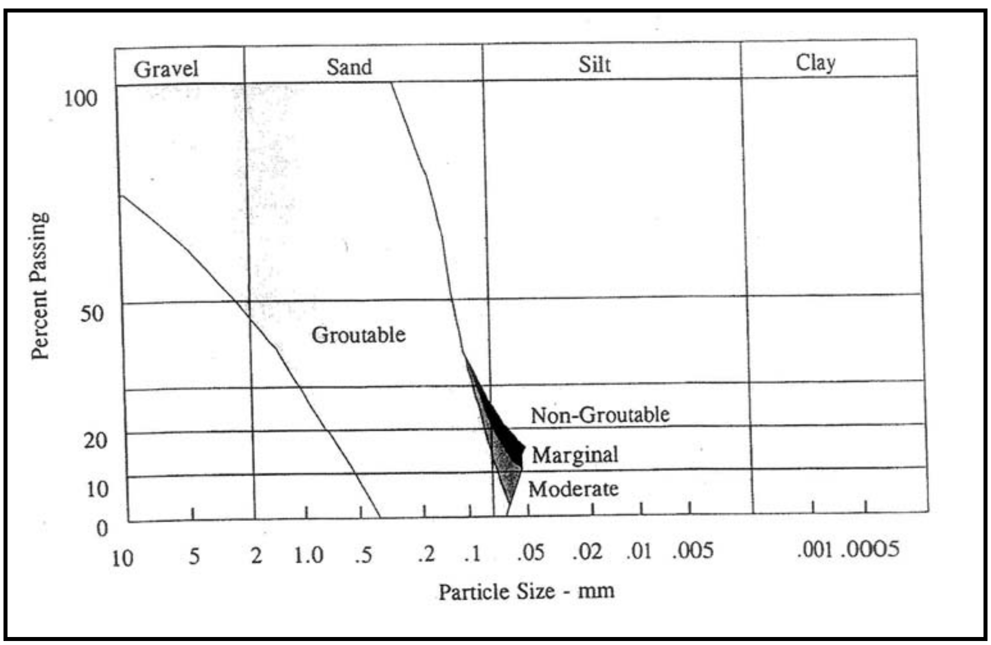

Figure 19-30, Grain-Size Distribution for Permeation Grouting (Ground Improvement Methods – August 2006)

These guidelines provide an indication of permeability; however, the actual permeability of a soil should be determined, either in the laboratory or in field pumping tests or injection tests. It should be noted that environmental permitting will be required for both pumping and injection testing. The following equations provide further guidance for the potential for permeation grouting using particulate grouts.

$$\frac{D_{15soil}}{D_{85grout}} = \Psi \qquad \text{Equation 19-23}$$

$$\frac{D_{10soil}}{D_{95grout}} = \Theta \qquad \text{Equation 19-24}$$

Where,

D_{15soil} = Diameter of the fifteen percent passing for soil
$D_{85grout}$ = Diameter of the eighty-five percent passing for the grout material
D_{10soil} = Diameter of the ten percent passing for soil
$D_{95grout}$ = Diameter of the ninety-five percent passing for the grout material

Table 19-25, Guide to Permeation Grout Potential

Groutability	Ψ	Θ
Impossible	< 11	< 6
Possible	11 – 24	6 – 11
Easy	> 24	>11

After a preliminarily determination that permeation grouting is feasible, an expert in the design of permeation grouting should be consulted to complete the final design.

19.8.3.2 Compaction Grouting

According to Ground Improvement Methods, “Compaction grouting features the use of low slump (usually 1 inch or less), low mobility grouts of high internal friction. In weak or loose soils, the grout typically forms a coherent ‘bulb’ at the tip of the injection pipe, thus compacting and/or densifying the surrounding soil. … If settlement has already occurred, careful compaction grouting may be used to lift and level any surface structures that have been impacted. Compaction grouts can be designed as an economic and controllable medium for helping to fill large voids, even in the presence of flowing water.”

Compaction grouting has a wide variety of applications, but is primarily used in South Carolina for soil densification (for both static and seismic enhancements) and for raising surficial structures. In soil densification applications, the soils should be free-draining, such as gravels, relatively clean sands and some coarser silts (see Figure 19-30). In fine-grained soils, pore pressures may not be able to dissipate and improvement may not be achievable. In these soils, compaction grouting may displace the soil, but not cause settlement or consolidation. The grout mix design is a critical part of compaction grouting; the grout must have a high internal friction and a low slump to ensure the “bulb” forms. There are no mathematical models for use in compaction grouting (i.e., establishing the spacing, rate of injection, limiting volumes, etc.). Therefore, either an engineer or contractor that specializes in compaction grouting should be retained to assist in the final design of compaction grouting. Typically compaction grout pipes are spaced at 6-1/2 to 16-1/2 feet intervals. The amount of grout required for soil densification ranges from three to twelve percent of the soil volume being treated. Normally, compaction grouts use particulate grouts such as Portland Cement Types I or II. The slump of the compaction grout should be around 1 inch.

19.8.3.3 Jet Grouting

Jet grouting is a grouting process that uses high pressure, high velocity erosive jets of water and/or grout to remove some of the soil and replacing the removed soil with cement based grout. The combination soil and grout is called “soilcrete”. Jet grouting can be used in soils ranging from clays to gravels with varying degrees of effectiveness. Jet grouting can be used for a variety of applications:

- Water Control
- Settlement Control
- Underpinning
- Scour Protection
- Excavation Support
- Liquefaction Mitigation
- Treatment of Karst

Jet grouting permits the shape, size and properties of treated soil, usually a circular column, to be engineered in advance. Figure 19-31 provides a schematic of the jet grouting procedure.

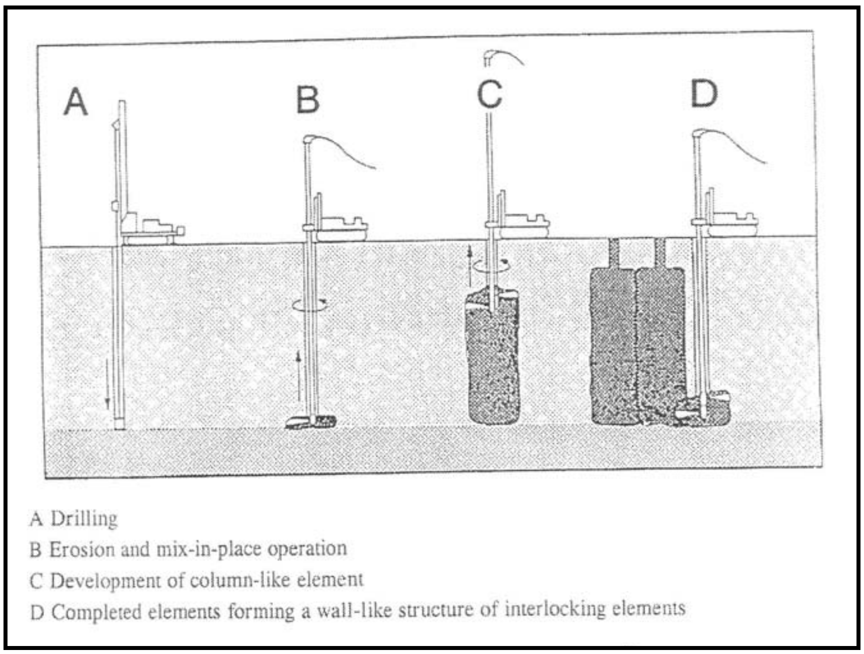

Figure 19-31, Jet Grouting Process Schematic
(Ground Improvement Methods – August 2006)

Jet grouting can be accomplished using three different types of jetting procedures as discussed below and depicted in Figure 19-32.

- Single Fluid System – The fluid is the grout and uses a high-pressure (7,200 psi) jet to simultaneously erode the in-situ soil and inject the grout. This system only partially replaces the soil.
- Double Fluid System – A high-pressure grout jet is contained within a compressed air cone. This system produces a larger column diameter, provides a higher degree of soil replacement, although a lower strength "soilcrete" is created.
- Triple Fluid System – An upper jet of high-pressure (4,400 to 7,200 psi) water contained inside a cone of compressed air is used for excavation, with a lower jet injects grout, at a lower pressure, to replace the slurried soil.

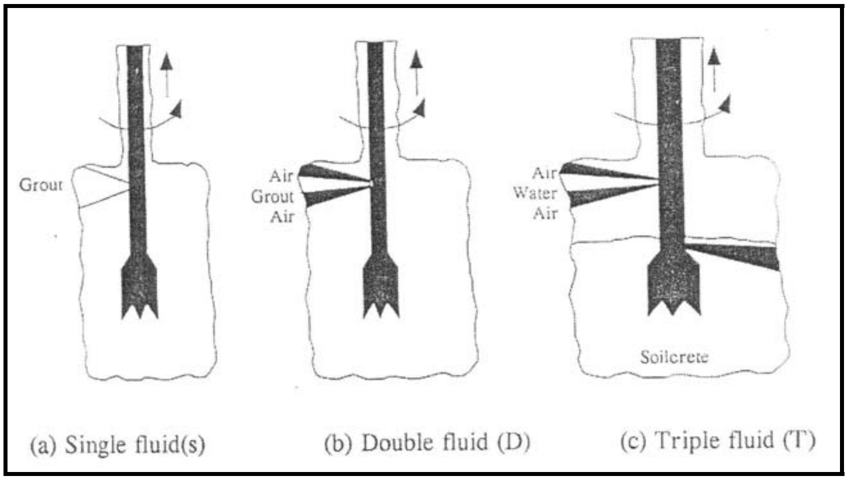

Figure 19-32, Jet Grouting Systems
(Ground Improvement Methods – August 2006)

19.8.3.4 Soil Fracture Grouting

Soil fracture grouting is the process of injecting grouts in a highly controlled manner that does not permit permeation of the grout in the soil matrix nor compaction of the soil matrix. Instead the soil matrix is ruptured and the grout forms a reinforcing "skeleton" within the matrix. Soil fracture grouting can be used to raise settled structures, control settlement, and soil reinforcement. Sophisticated measuring equipment is required when conducting this type of grouting operation. Similar to compaction grouting, designs using soil fracture grouting should be performed by an engineer or contractor specializing in this method.

19.9 COLUMN SUPPORTED EMBANKMENT

Constructing embankments over soft, compressible soils creates numerous problems (i.e., excessive settlements, embankment instability, and long times for settlements to occur). These problems have led to the development of the ground improvement methods discussed previously in this Chapter; however, in certain cases, time constraints are critical to the success of the project. Therefore, an alternate ground improvement method has been used: Column Supported Embankment (CSE) (see Figure 19-33). CSEs consist of two primary components; first, a column system to transfer loads to a more suitable bearing stratum and second, a load transfer platform (LTP). The LTP can consist of either structural concrete or a geosynthetic reinforced soil layer.

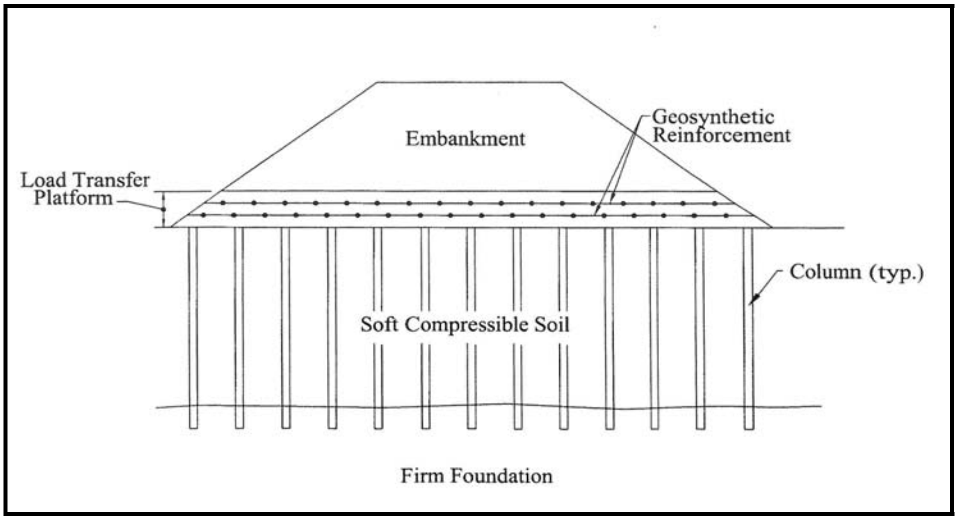

Figure 19-33, CSE with Geosynthetic LTP
(Ground Improvement Methods – August 2006)

The columns consist of typical deep foundation elements such as driven piling (prestressed concrete or steel H- or pipe piles or timber); however, the use of driven concrete or steel piling may not be economical since the capacity developed by these pile types would exceed the demand placed on the piles. These piling types should only be used if the LTP is composed of structural concrete. If the LTP is a geosynthetic reinforced soil layer, concrete and steel piling should not be used. Other types of columns can consist of timber piling, stone columns, geotextile encased columns, vibro-concrete columns, Geopiers, soil mixed columns, or continuous flight auger piles.

The LTP transfers the embankment load to the columns. The LTP may consist of either a rigid structural element or a geosynthetic reinforced soil layer. The rigid LTP is typically economically cost prohibitive and will therefore, not be discussed in this Chapter. The use of a LTP allows for the columns to be more widely spaced; however, the use of an LTP is not required if the columns are closely spaced, see Figure 19-34.

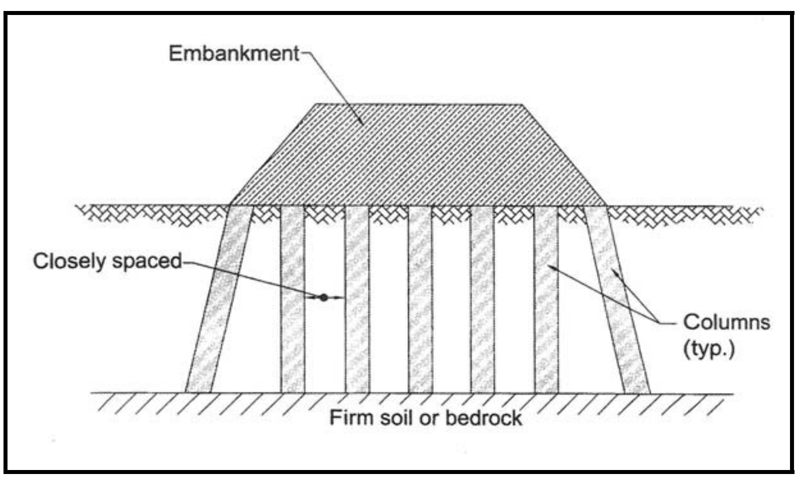

Figure 19-34, CSE without LTP
(Ground Improvement Methods – August 2006)

19.9.1 Analysis and Preliminary Design

As indicated previously, CSEs have traditionally been used to support embankments over soft soils when time constraints are such that consolidation of the soft soils is not practical. CSEs have the advantage of being constructed in a single stage. There is no waiting period for the dissipation of pore water pressures. CSEs are more economical than removing and replacing the soil, especially when the groundwater is close to the ground surface. Where infrastructure precludes high-vibration techniques, the type of column used for the CSE system may be selected to minimize or eliminate the potential for vibrations. Total and differential settlement of the embankment may be drastically reduced when using CSEs over other conventional approaches. Another benefit of using CSEs is that a variety of columns are available for support of the embankment depending on the stiffness of the subsurface soils. CSEs have the major disadvantage of having high initial costs; however, the savings in time can offset these costs. An additional disadvantage of CSEs is there is no single accepted design method. There are multiple methods and all provide different answers.

Typically, CSEs have been limited to embankment heights of approximately 35 feet. The thickness of the soft soil is not a critical component in the determination of the feasibility of using CSEs because there are a variety of columns that can be used for support. The determination of the feasibility of using CSEs should consider the following factors:

- The preliminary spacing of the columns should be limited so that the area replacement ratio is between ten and twenty percent.
- The clear span between columns should be less than the embankment height and should not exceed approximately 10 feet. Wider clear spans may lead to unacceptable differential settlement between columns.
- The fill required to create the LTP shall be select structural fill with an effective friction angle greater than or equal to 35°.
- The columns shall be designed to carry the entire load of the embankment.
- The CSE reduces post construction settlements of the embankment surface to typically less than 2 to 4 inches.

The selection of the column should also consider the potential environmental impact of the installation of the column.

19.9.2 Design

The design of CSEs is a complicated soil-structure interaction problem that requires the engineer to have a good understanding of the Strength and Service limit states of the structure. All of the design methods currently in use are empirical and primarily focus on the design of the LTP. These empirical methods include:

- The British Standard (BC8006)
- The Swedish Standard
- The German Method
- The Collin Method

The Strength limit state failure modes include the following (see Figure 19-35):

a. Failure of the columns to carry the full embankment load
b. The lateral extent of the columns must be sufficient to prevent slope instability
c. The load transfer platform must be designed to transfer the vertical load to the columns
d. Lateral sliding of the embankment on top of the columns
e. The global (overall) stability must be checked

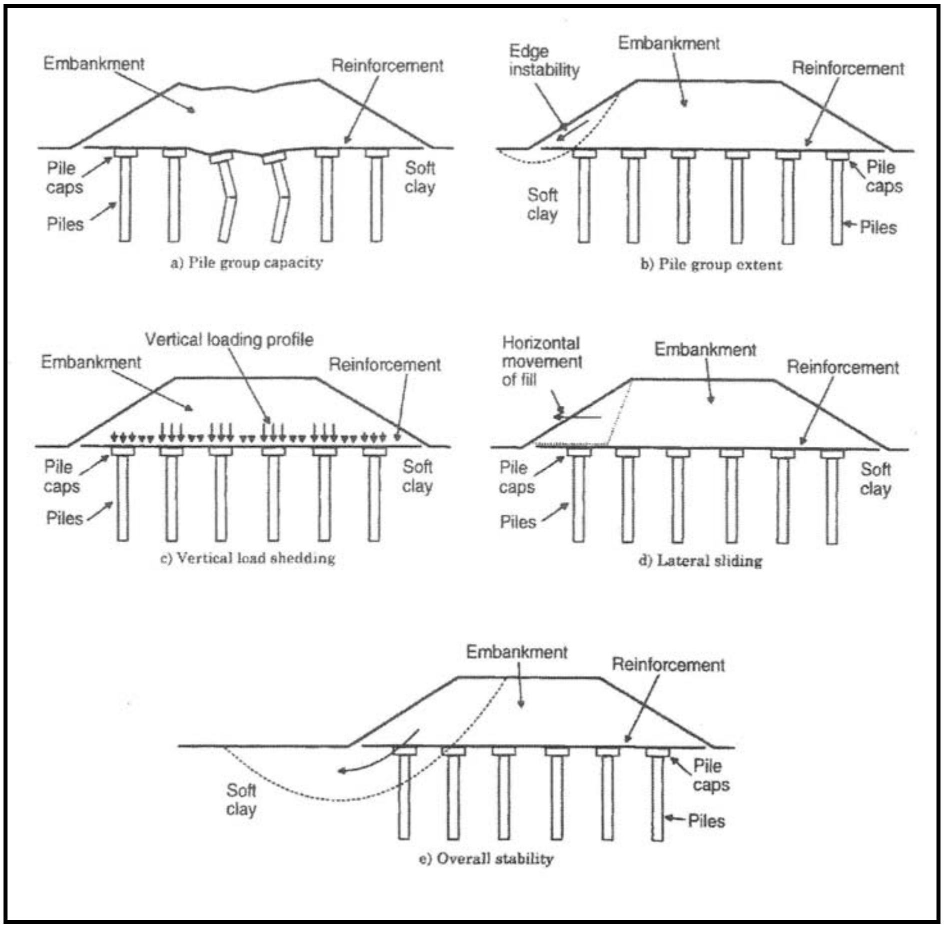

Figure 19-35, Strength Limit State Failure Modes (Ground Improvement Methods – August 2006)

The Service limit state of the CSE must also be checked. The strain in the geosynthetic reinforcement used to create the LTP should be kept below some maximum threshold to preclude unacceptable deformation reflection (see Figure 19-36, Detail a) at the top of the embankment. In addition, the settlement of the columns should also be analyzed to ascertain whether the CSE will develop unacceptable settlements (see Figure 19-36, Detail b).

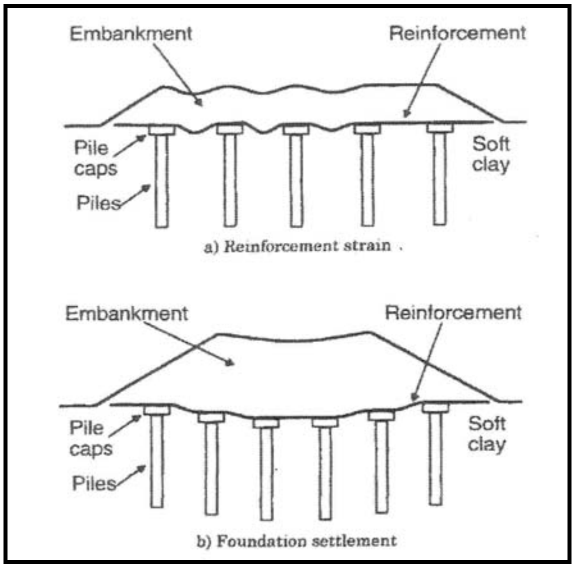

Figure 19-36, Service Limit State
(Ground Improvement Methods – August 2006)

The general design procedure for CSEs is provided below:

1. Estimate preliminary column spacing (see previous Section)
2. Determine required column load
3. Select preliminary column type based on required column load and site geotechnical requirements
4. Determine capacity of column to satisfy Strength and Service limit state design requirements
5. Determine extent of columns required across embankment width
6. Select LTP design approach (catenary or beam)
7. Determine reinforcement requirements based on estimated column spacing.
8. Revise column spacing as required.
9. Determine reinforcement requirements for lateral spreading.
10. Determine overall reinforcement requirements based on LTP and lateral spreading.
11. Check global stability.
12. Prepare construction drawings and specifications.

19.9.2.1 Column Design

The selection of the type of column should be based on the constructability, load capacity, and cost of the various column types. The load carrying capacity of each column is based on the tributary area of each column (see Figure 19-37). In CSE design, it is assumed that the weight of the embankment and any surcharge loads are carried by the columns and that the surrounding soil carries minimal, if any, load. The tributary area for a single column is geometrically a hexagon; however, for simplification a circle having the same tributary area is used. Figure 19-37 provides the effective diameter (D_e) for both triangular and square spacings. The typical center-to-center column spacing is 5 to 10 feet. The required design vertical load (Q_r) in the column is determined by the following equation:

$$Q_r = \pi\left(\frac{D_e}{2}\right)^2(\gamma H + q)$$ Equation 19-25

Where,

D_e = Effective tributary area of column
H = Height of embankment
γ = Unit weight of embankment soil
q = Live and dead load surcharge (determined similar to long-term stability analysis)

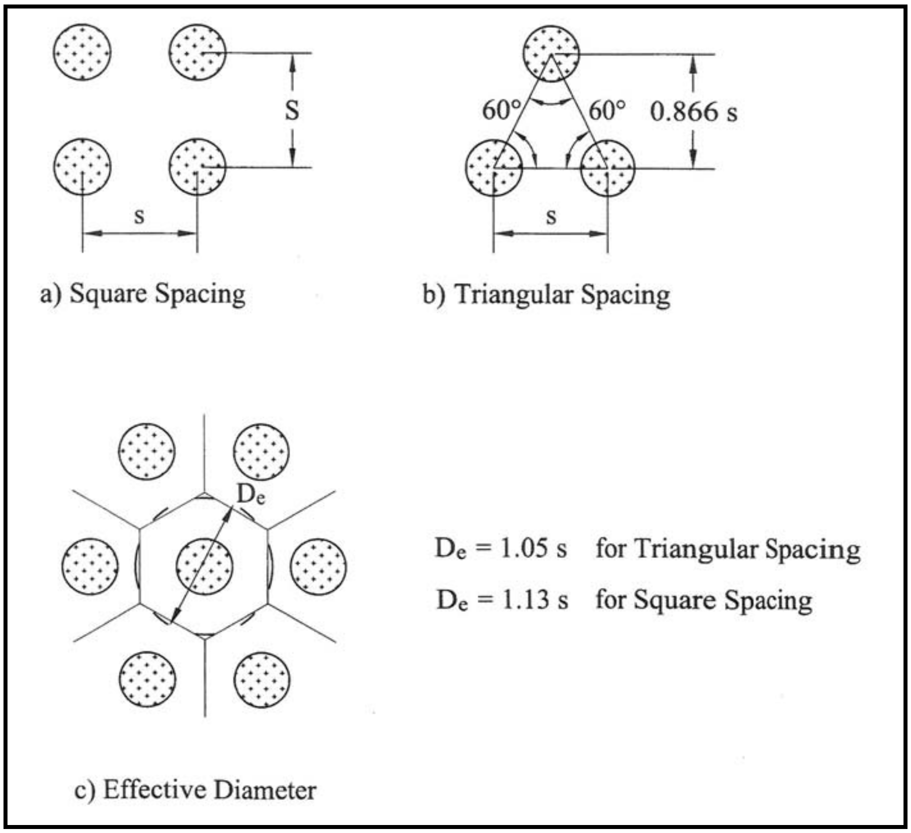

Figure 19-37, CSE Column Layout
(Ground Improvement Methods – August 2006)

In addition to determining the load to be carried by the columns, the lateral extent of the columns will also need to be determined. The columns should extend a sufficient distance beyond the crest of the embankment to ensure that any instability or differential settlement that occurs beyond the limits of the columns will not affect the crest of the embankment. The extent of the columns should be determined using a slope stability program (see Chapter 17). For preliminary designs and feasibility studies, the following equations from the British Standard (BS8006) may be used.

$$L_p = H(n - \tan\theta_p)$$ Equation 19-26

$$\theta_p = \left(45 - \frac{\varphi'_{emb}}{2}\right)$$ Equation 19-27

Where,

L_p = Horizontal distance from the toe of the embankment to the edge of first column

n = Side slope of embankment (see Figure 19-38)

θ_p = Angle from vertical between the outer-most column and the crest of the embankment (see Figure 19-38)

φ'_{emb} = Effective friction angle of embankment fill

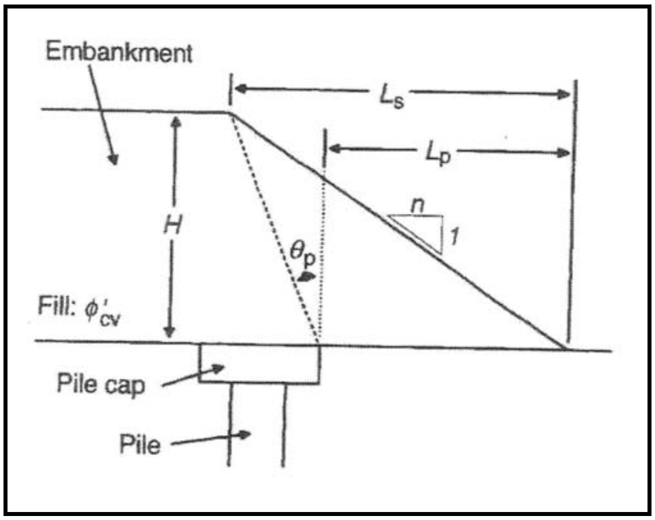

Figure 19-38, CSE Edge Stability
(Ground Improvement Methods – August 2006)

The potential for lateral spreading of the embankment must be analyzed (Figure 19-39). The geosynthetic reinforcement must be designed to prevent lateral spreading of the embankment. This is a critical aspect of the design, because many columns used to support CSEs are not capable of developing adequate lateral resistance to prevent the spreading of the embankment. The geosynthetic reinforcement must be designed to resist the horizontal force caused by the lateral spreading of the embankment. The required tensile force to prevent lateral spreading (T_{ls}) is determined using the following equations.

$$T_{ls} = \frac{K_a(\gamma H + q)H}{2}$$ Equation 19-28

$$K_a = \tan^2\left(45 - \frac{\varphi'_{emb}}{2}\right)$$ Equation 19-29

The minimum length of reinforcement (L_e) required to prevent the sliding of the embankment across the reinforcement is determined using the following equation.

$$L_e = \frac{T_{ls}}{[0.5\gamma H(c_{iemb}\tan\varphi'_{emb})]}$$ Equation 19-30

Where,

c_{iemb} = Coefficient of interaction for sliding between the geosynthetic reinforcement and the embankment fill

φ'_{emb} = Friction angle of embankment fill material (φ'_{cv} in figure 19-39

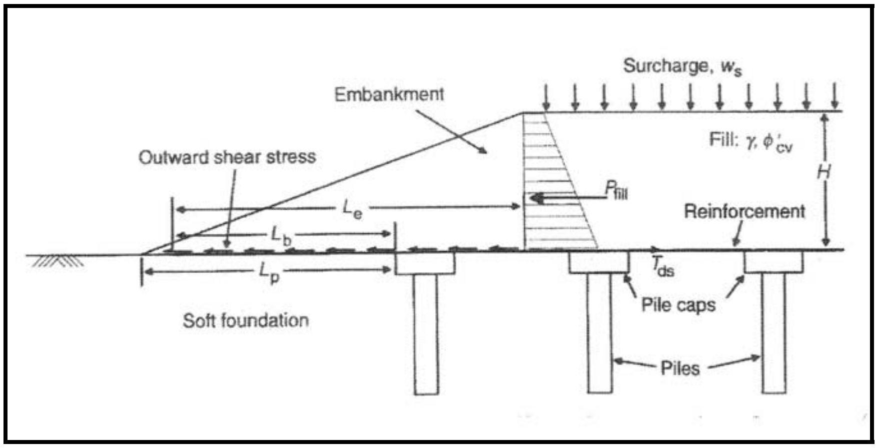

Figure 19-39, CSE Lateral Spreading
(Ground Improvement Methods – August 2006)

19.9.2.2 Load Transfer Platform

The load transfer platform (LTP) has two design approaches (catenary or beam) (see Figure 19-40). The three methods for design of the LTP using the catenary design approach are the British Standard (BS 8006), the Swedish Method and the German Method. The catenary approach makes the following assumptions:

- Soil arch forms in the embankment
- Reinforcement is deformed during loading
- One layer of reinforcement is used

The beam design approach has one design method, the Collin. The beam approach makes the following assumptions

- A minimum of three layers of reinforcement are used to create the platform
- Spacing between the layers of reinforcement is 8 to 18 inches
- The platform thickness is greater than or equal to one-half of the clear span between columns (edge-to-edge)
- Soil arch is fully developed with the depth of the platform

The catenary approach normally requires higher strength reinforcement for the same design conditions, as opposed to the beam approach. The beam approach will typically allow for larger column-to-column spacing than the catenary approach for standard geosynthetics (i.e., materials available off the shelf).

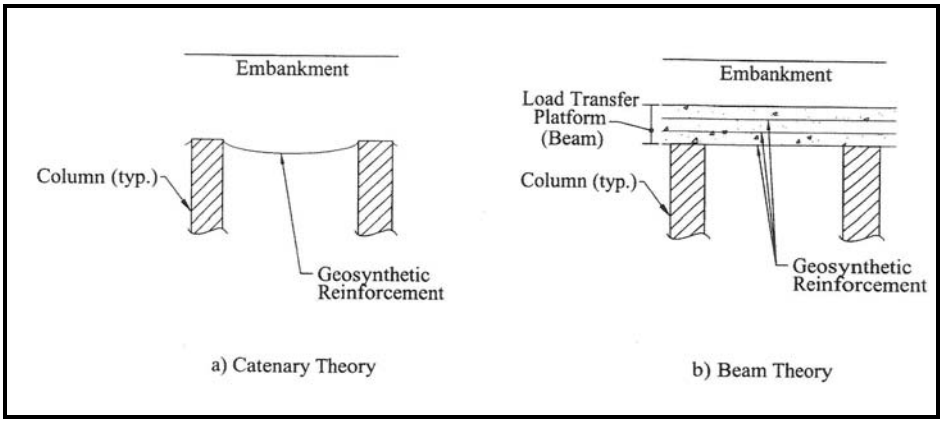

Figure 19-40, Load Transfer Mechanisms
(Ground Improvement Methods – August 2006)

Both design approaches rely on the soil forming an arch. Soil arching, according to Ground Improvement Methods, is "the ability of material to transfer loads from one location to another in response to a relative displacement between locations." Figure 19-41 provides schematics of soil arching.

According to Ground Improvement Methods, "The stress at point "a" in Figure *19-41, Detail* a, is equal to the overburden stress γH, where γ is the unit weight of the soil, and H is the height of the soils mass. At the moment when the soil loses support, a temporary true arch is formed. The soil at point "a" is in tension, and the weight of the soil prism starts to be transferred to the adjacent unyielding soil (Figure *19-41, Detail* b). Deformation within the temporary true soil arch occurs. As the soil settles into an inverted arch (Figure *19-41, Detail* c), an equilibrium state is achieved, the adjacent unyielding soil mobilizes its shear strength, and the load transfer is complete. At some height (H_e) above point "a," the transfer of stress is complete. The settlements in the soil mass above this point are uniform. The degree of soil arching is defined as the soil arch ratio (ρ), which is the ratio of the average vertical stress on the yielding portion (i.e., soft soil between columns) to the average vertical stress due to the embankment fill and surcharge load."

$$\rho = \frac{\sigma_s}{(\gamma_{emb} H + q)} \qquad \text{Equation 19-31}$$

Where,

H = Height of embankment

γ_{emb} = Unit weight of embankment material

q = Live and dead load surcharge (determined similar to long-term stability analysis)

σ_s = Average vertical stress applied between columns

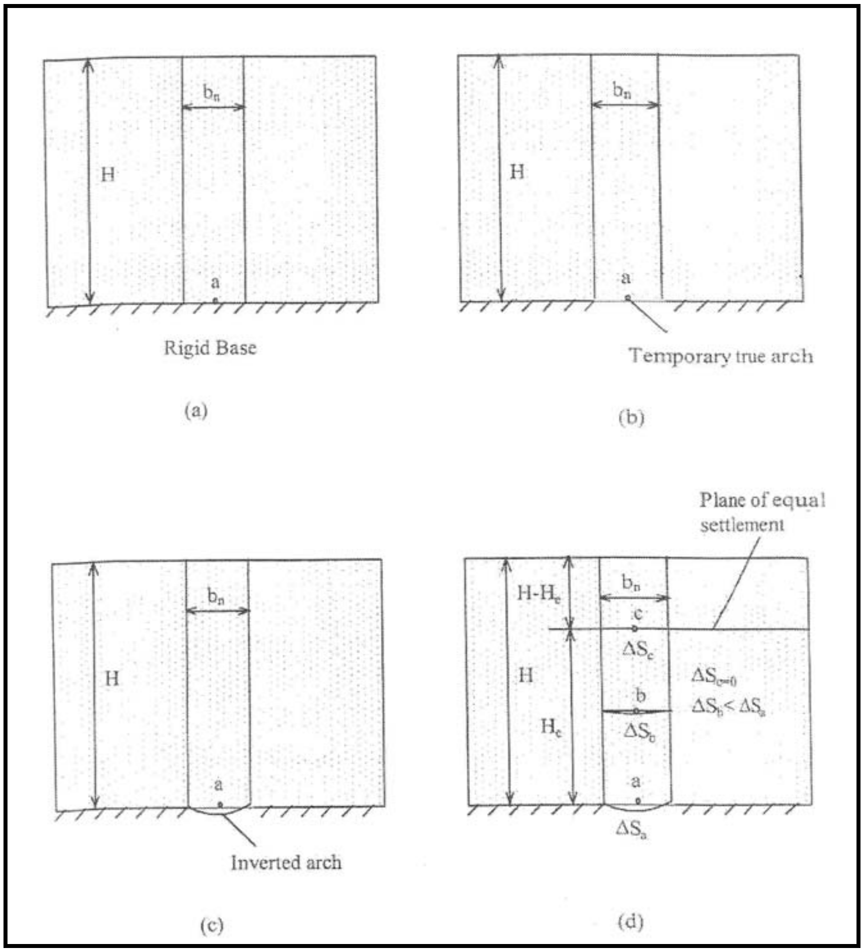

Figure 19-41, Soil Arching
(Ground Improvement Methods – August 2006)

In addition to soil arching, the load transfer platform design relies on the geosynthetic developing resistance to tension (i.e., tension membrane theory). The vertical load from the soil within the arch and any surcharge load, if the thickness of the embankment is not great enough to develop the full arch, is carried by the reinforcement.

The variables and symbols used by all of the design methods have been standardized. Figure 19-42 depicts the common symbols that will be used in presenting each of these methods; further each variable and/or symbol is defined also.

Where,

d = Diameter of column
H = Height of embankment
P_c' = Vertical stress on column
q = Surcharge load
s = Center-to-center column spacing
T_{RP} = Tension in the extensible reinforcement
W_T = Vertical load carried by the reinforcement
γ_{emb} = Unit weight of embankment material
φ'_{emb} = Effective friction angle of the embankment fill material

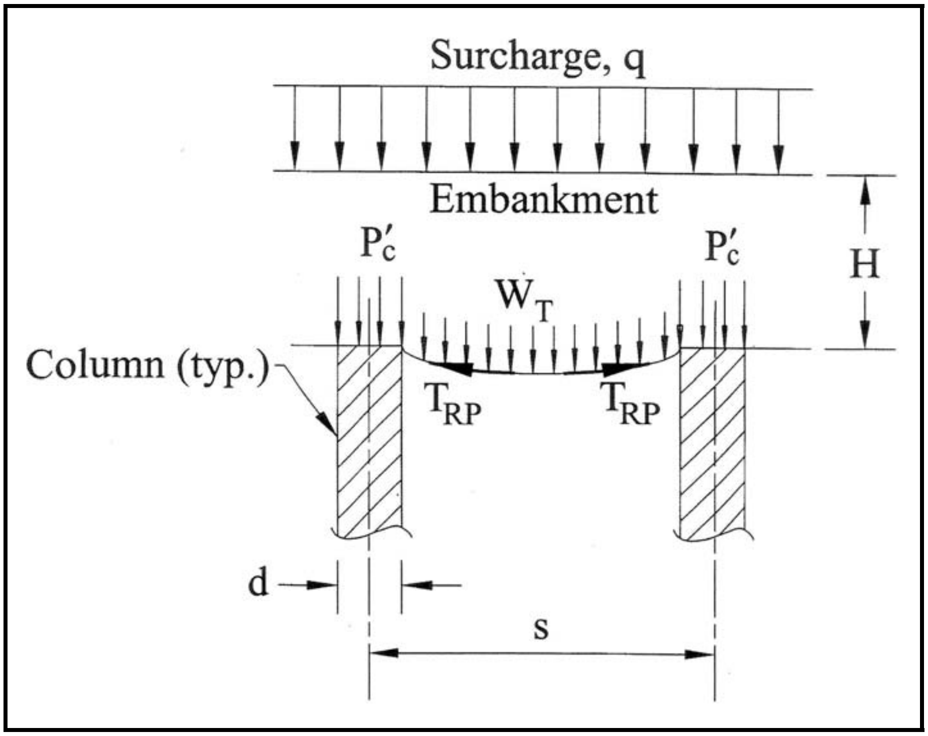

Figure 19-42, CSE Definition of Terms
(Ground Improvement Methods – August 2006)

19.9.3 Catenary Design Approach

The catenary design approach depends on the arching effect of the soil and the ability of the geosynthetic reinforcement to resist in tension the load applied by the embankment and surcharge. There are three design methods that use the catenary design approach:

- British Standard (BS 8006)
- Swedish Method
- German Method

19.9.3.1 British Standard (BS 8006)

To ensure that differential settlement does not occur at the surface of the embankment, the British Standard recommends that the embankment height be a minimum of 1.4 times the clear span (edge-to-edge) between columns. At this height, the soil arches and the columns carry more load than the surrounding soil. The ratio of vertical stress on the columns to the average vertical stress at the base of the embankment is determined from the following equations.

$$\frac{P'_c}{\sigma'_c} = \left[\frac{C_c d}{H}\right]^2$$ Equation 19-32

$$\sigma'_c = \left(f_{fs}\gamma_{emb}H + f_q q\right)$$ Equation 19-33

Where,

σ'_c = Average vertical stress at the base of the embankments
f_{fs} = Partial soil unit mass load factor (1.3)
f_q = Partial surcharge load factor (1.3)
C_c = Arching coefficient
= [(1.95H/d)-0.18] for end bearing columns (unyielding)
= [(1.50H/d)-0-07] for frictional columns (normal)

The vertical load (W_T) carried by the reinforcement is determined from the following equations:

Table 19-26, Vertical Load Determination Equations

H	W_T	
< 0.7 (s-d)	N/A	
0.7 (s-d) ≤ to ≤ 1.4 (s-d)	$\left[\frac{s(f_{fs}\gamma H + f_q q)}{(s^2 - d^2)}\right]\left[s^2 - d^2\left(\frac{P'_c}{\sigma'_c}\right)\right]$	Equation 19-34
> 1.4 (s-d)	$\left[\frac{1.4sf_{fs}\gamma(s - d)}{(s^2 - d^2)}\right]\left[s^2 - d^2\left(\frac{P'_c}{\sigma'_c}\right)\right]$	Equation 19-35

The tension in extensible reinforcement (T_{RP}) per linear foot of reinforcement resulting from the distributed load is determined using the following equation.

$$T_{RP} = 0.5W_T\left[\frac{(s-d)}{d}\right]\sqrt{\left(1+\frac{1}{6\varepsilon}\right)}$$ Equation 19-36

Where,

ε = Strain in the reinforcement

Some initial strain is required to generate a tensile force in the reinforcement; however, the practical upper limit on strain is six percent. Using this limit ensures that the embankment load is transferred to the columns. The long-term strain in the reinforcement (caused by creep) should be limited to ensure that the long-term localized deformations (dimples) do not occur at the ground surface. A minimum creep strain of two percent over the design life (100 years) of the reinforcement is allowed.

19.9.3.2 Swedish Method

The Swedish Method has many similarities to the British Standard and is valid when the following assumptions/parameters are satisfied:

- Arch formation occurs
- Reinforcement is deformed during loading
- One layer of reinforcement is used
- Reinforcement is located within 4 inches of the column
- The embankment height is greater than or equal to the clear distance between the columns (edge-to-edge)

- The ratio of column area to the influence area per column is greater than or equal to ten percent
- The embankment fill effective friction angle is 35°
- Initial strain in the reinforcement is limited to six percent
- Long-term (creep) strain is limited to two percent
- Total strain is less than seventy percent strain at failure

The model used in the Swedish Method to determine the vertical load is provided in Figure 19-43. The cross sectional area of the soil under the arch, which is the load carried by the reinforcement, is approximated by the soil wedge shown in Figure 19-43. This applies even when the embankment height is lower than the top of the soil wedge. The height of the soil wedge is determined from the following equation:

$$h = \frac{(s-d)}{2\tan 15^\circ} \qquad \text{Equation 19-37}$$

Where,

h = Height of soil wedge or arch
s = Center-to-center column spacing
d = Diameter of column

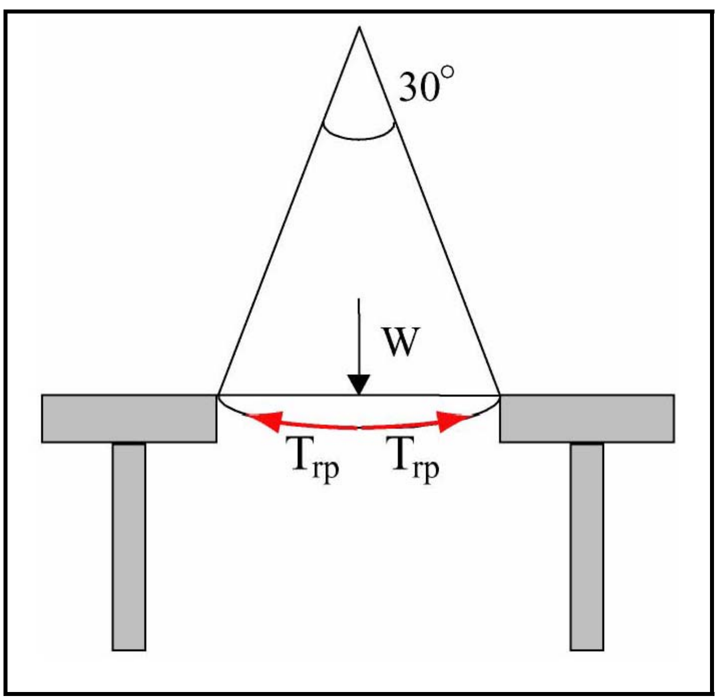

Figure 19-43, Swedish Method Model
(Nordic Guidelines for Reinforced Soils and Fills – February 2004)

The two-dimensional weight (W_T) of the soil wedge per unit length in depth is determined from the following equation.

$$W_T = \frac{(s-d)^2 \gamma_{emb}}{4\tan 15^\circ} \qquad \text{Equation 19-38}$$

The force in the reinforcement, per unit length in depth, due to the two-dimensional weight (W_T) in three-dimensions is determined using the following equation.

$$T_{RP} = 0.5W_T\left[1+\left(\frac{s}{d}\right)\right]\sqrt{1+\frac{1}{6\varepsilon}} \qquad \text{Equation 19-39}$$

19.9.3.3 German Method

The German Method considers the effect of the soft foundation soil in determining the load carrying capability of the reinforcement, unlike either the British Standard or Swedish Methods, which do not. The German Method considers both the undrained shear strength of the foundation material, as well as the shear strength of the embankment material. This method is only applicable if the height of the embankment is greater than the column spacing. Two failure criteria are considered:

1. Failure of the embankment fill at the crown of the arch
2. Failure at the bearing point of the arch

The ratio (E) of the vertical load on the columns to the average load at subgrade is a function of which failure mode controls the design. Failure at the crown of the arch occurs for relatively shallow embankments with wide column spacing. E is determined from the following equations:

$$E = 1-\left[1-\left(\frac{d}{s}\right)^2\right](A - AB + C) \qquad \text{Equation 19-40}$$

$$A = \left[1-\left(\frac{d}{s}\right)\right]^{2(K_p-1)} \qquad \text{Equation 19-41}$$

$$B = \left[\frac{s}{1.41H}\right]\left[\frac{(2K_p-2)}{(2K_p-3)}\right] \qquad \text{Equation 19-42}$$

$$C = \left[\frac{(s-d)}{1.41H}\right]\left[\frac{(2K_p-2)}{(2K_p-3)}\right] \qquad \text{Equation 19-43}$$

$$K_p = \frac{(1+\sin\varphi'_{emb})}{(1-\sin\varphi'_{emb})} = \tan^2\left(45+\frac{\varphi_{emb}'}{2}\right) \qquad \text{Equation 19-44}$$

Failure at the bottom of the arch is determined using the following equations.

$$E = \frac{\beta}{(1+\beta)} \qquad \text{Equation 19-45}$$

$$\beta = \left[\frac{2K_p}{(K_p+1)\left(1+\frac{d}{s}\right)}\right]\left[\left(1-\frac{d}{s}\right)^{-K_p} - \left(1+\frac{K_p d}{s}\right)\right] \qquad \text{Equation 19-46}$$

The minimum value of E controls the stress applied to the soil between the columns (σ_s). The stress that is applied to the soil between columns (Figure 19-44) is determined using the following equation.

$$\sigma_s = \left[\frac{(\gamma_{emb}H + q)}{(s^2 - d^2)}\right][1-E]s^2 \qquad \text{Equation 19-47}$$

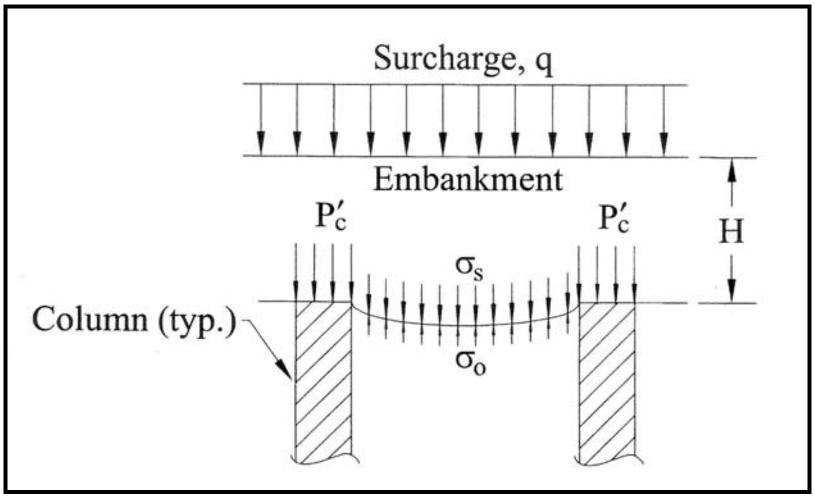

Figure 19-44, German Method Model
(Ground Improvement Methods – August 2006)

The geosynthetic reinforcement carries the stress imposed by the embankment (σ_s) minus the resistance of the soil (σ_o) located between the columns. The resistance of the soil (σ_o) between the columns is determined using the following equation.

$$\sigma_o = 0.5[(2+\pi)c_u] \qquad \text{Equation 19-48}$$

Where,

c_u = Undrained shear strength of the soil between the columns

0.5 = Resistance Factor (φ) used to determine σ_o

The vertical load (W_T) per unit length on the geosynthetic reinforcement spanning between the columns is determined using the following equation.

$$W_T = \left[\frac{\sigma_s(s^2-d^2)}{2(s'-d)}\right] - \left[\frac{\sigma_o(s^2-d^2)}{2(s'-d)}\right] \qquad \text{Equation 19-49}$$

Where,

s' = s for square column pattern
s' = 1.4s for triangular column pattern

The German Method assumes that the geosynthetic reinforcement is placed less than 1-1/2 feet above the columns. The tensile force in the reinforcement per unit length of reinforcement is determined based on catenary tension and is determined using the following equation.

$$T_{RP} = W_T \left[\frac{(s'-d)}{2d} \right] \left(\sqrt{1 + \frac{1}{6\varepsilon}} \right) \qquad \text{Equation 19-50}$$

19.9.4 Beam Design Approach

The beam design approach consists of one method, the Collin Method, and is fundamentally different from the catenary design approach. The beam design approach is based on the premise that the reinforcement creates a stiffened beam of reinforced soil to distribute the load imposed by the embankment to the columns. The stiffened beam of reinforced soil should contain a minimum of three layers of reinforcement (Figure 19-45). This beam acts as the LTP for the Collin Method.

The Collin Method is based on the following assumptions:

- The thickness (h) of the LTP is equal to or greater than one-half of the clear span between the columns (i.e., 0.5(s-d))
- A minimum of three layers of geosynthetic reinforcement is used to create the LTP
- A minimum distance of 8 inches is maintained between the layers of reinforcement
- Select fill is used to construct the LTP
- The primary function of the reinforcement is to provide lateral confinement of the select fill to facilitate soil arching within the thickness (h) of the LTP
- The secondary function of the reinforcement is to support the wedge of the soil below the arch
- All of the vertical load from the embankment above the load transfer platform is transferred to the columns below the platform
- The initial strain in the reinforcement is limited to five percent

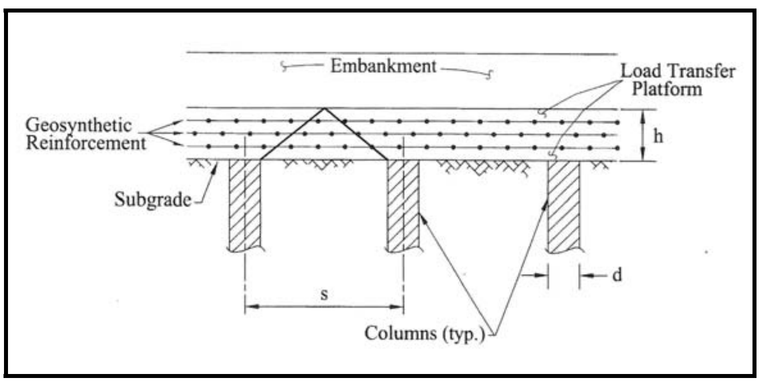

Figure 19-45, Load Transfer Platform
(Ground Improvement Methods – August 2006)

The fill load attributed to each layer of reinforcement is the material located between the layer of reinforcement and the next layer above (Figure 19-46). The uniform vertical load on any layer (n) of reinforcement (W_{Tn}) may be determined using the following equations for a triangular pattern and a square pattern, respectively.

$$W_{Tn} = \frac{\left((s-d)_n^2 + (s-d)_{n+1}^2\right)\sin 60^\circ h_n \gamma_{emb}}{(s-d)_n^2 \sin 60^\circ} \quad \text{Equation 19-51}$$

$$W_{Tn} = \frac{\left((s-d)_n^2 + (s-d)_{n+1}^2\right) h_n \gamma_{emb}}{(s-d)_n^2} \quad \text{Equation 19-52}$$

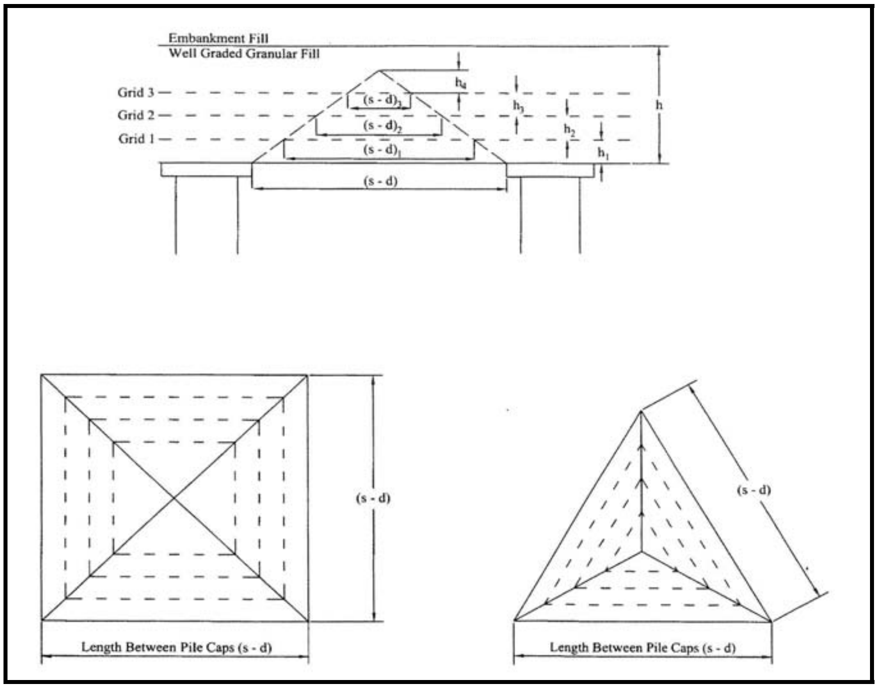

Figure 19-46, Collin Method Load Transfer Platform Design (Ground Improvement Methods – August 2006)

The tensile load on any layer of reinforcement (T_{RPn}) is determined based on tension membrane theory and is a function of the amount of strain in the reinforcement. T_{RPn} is determined using the following equation:

$$T_{RPn} = W_{Tn} \Omega D / 2 \quad \text{Equation 19-53}$$

Where,

D = $(s\text{-}d)_n$ for square column spacing
D = $(s\text{-}d)_n$tan30 for triangular column spacing
Ω = From Table 19-27

Table 19-27, Values of Ω
(Ground Improvement Methods – August 2006)

Ω	Reinforcement Strain (ε)%
2.07	1
1.47	2
1.23	3
1.08	4
0.97	5

19.9.5 Reinforcement Total Design Load

Regardless of the method used to design the LTP, the maximum design load (T_{max}) on the geosynthetic reinforcement is determined using the following equations:

Reinforcement along the length of the embankment (longitudinal direction of road)

$$T_{max} = T_{RP} \qquad \text{Equation 19-54}$$

Reinforcement across the width of the embankment (transverse direction of road)

$$T_{max} = T_{RP} + T_{ls} \qquad \text{Equation 19-55}$$

19.10 REINFORCED EMBANKMENTS ON SOFT FOUNDATIONS

Embankments constructed on soft soil foundations have a tendency to move both in the vertical as well as the horizontal directions. The vertical settlements are dealt with using ground improvement methods discussed previously in this Chapter. The horizontal movements can consist of either a general sliding of the embankment (block type failure) or from lateral squeeze (see Chapter 17). As indicated in Chapter 17, the soft soils will gain strength with time due to the settlement, however, some reinforcement of the subgrade may be required to prevent lateral movements or slope instabilities while the subgrade soils are gaining strength. There are two uses of reinforced embankments on soft soil foundations. The first is as an aide to construction and the second is reinforcement of the slope.

The use of reinforced embankments over soft foundation soils typically fall into two situations; first, construction over uniform deposits, and second, construction over localized anomalies (Figure 19-47). The most common application in transportation construction is the placement of embankments over uniform soft soil foundations. Typically, the reinforcement is placed perpendicular to the centerline of the embankment to prevent long joints parallel to the centerline and the potential for sliding of the outer most reinforcement. As the end of the embankment is approached, the turning of the reinforcement may be required.

The reinforcement normally used consists of biaxial and uniaxial geogrids; however, geotextiles may also be used. The design using geotextiles is based upon constructability, survivability, and the amount of strain required to achieve the desired strength.

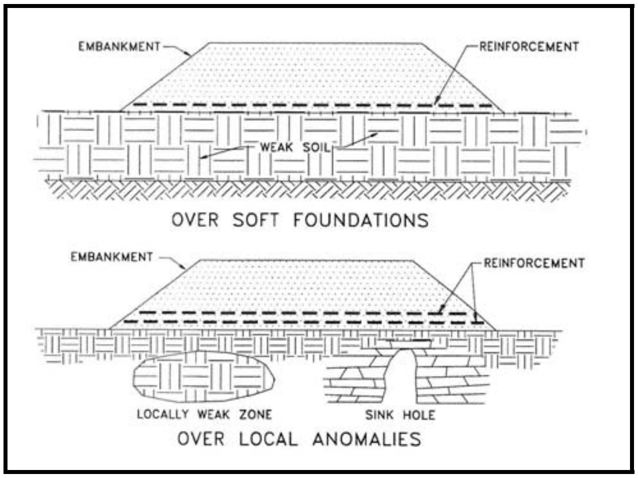

Figure 19-47, Reinforced Embankment Applications (Ground Improvement Methods – August 2006)

19.10.1 Subgrade Stabilization

The use of reinforcement beneath an embankment as subgrade stabilization is also called "bridging". Bridging is only required if the in-situ soil has an undrained shear strength ($\tau = c_u$) less than 500 pounds per square foot or 3.5 pounds per square inch. A bridge lift should be considered if the exposed subgrade soils are susceptible to deterioration (i.e. contain plastic fines) from inclement weather and exposure to vehicular traffic. Basically, the reinforcement is not considered as part of the design of the embankment, but is placed exclusively to permit construction to proceed, by stabilizing the subgrade materials to permit the placement of bridging materials. Further, the use of reinforcement and bridging materials will not prevent or mitigate settlement or slope instability; other ground improvement methods are required to mitigate settlement or slope instability. The reinforcement typically consists of either a geogrid or a geotextile. The use of the reinforcement is to limit the amount of excavation (undercutting or mucking) required. The standard construction practice using reinforcement to aide construction is presented below.

1. Muck excavation to required depth (if necessary)
2. Placement of reinforcement and/or soil separator (if necessary)
3. Placement of bridge lift
4. Placement of soil separator (if necessary)
5. Placement and compaction of backfill materials

The bridge lift may consist of either stone (No. 57 or No. 67) or granular materials (A-1-a, A-1-b, A-3, A-2-4, A-2-5, A-2-6) and is not normally compacted to the level required for the remainder of the embankment. Bridge lift materials placed in water should consist of stone or coarse granular materials (A-1-a). If mucking is performed below the water level, the bridge lift is normally placed to at least 6 inches above high water, regardless of the bridge thickness determined; a soil separator is placed on top of the bridge lift as necessary. The thickness of the bridge lift is determined using both the US Forest Service (Steward, et. al.) and the Giroud-Han methods as presented in Geosynthetic Design and Construction Guidelines. The thickest bridge lift shall be used in design. The top of the bridge lift shall not be closer than 3 feet beneath the bottom of the pavement structure, unless the bridge lift is constructed of stone.

19.10.1.1 US Forest Service (Steward, et. al.) Method

The US Forest Service (USFS) Method is a chart based solution that requires knowledge of not only the soil conditions, but the methods of fill placement and sizes of construction equipment. Since it will be practically impossible to ascertain the type and size of construction equipment to be used, the type and size of construction equipment should be indicated on the drawings as a limitation until at least 3 feet of embankment fill has been placed. This method is applicable to both geotextiles and geogrids.

The first step in using the USFS Method is determining the subgrade strength, the undrained shear strength (c_u, τ (psi)) should be determined from either CPT or DMT soundings or from field vane shear tests. Undrained shear strength, in psi, may also be estimated from field CBR values using the following equation.

$$c_u = 4.3(CBR) \qquad \text{Equation 19-56}$$

The second and third steps handle the anticipated traffic configuration. The type of construction equipment anticipated should be indicated as well as the amount traffic passes. It should be noted that the minimum number of traffic passes is 100, while the maximum is 1,000. It should be noted that the traffic estimate is based on the vehicles having a tire pressure of 80 psi. In the fourth step the depth of the tolerable rut is determined. The depth of the tolerable rut ranges from 2 to 4 inches.

The fifth step is determining the bearing capacity factor (N_c) for both the condition of without reinforcement and with reinforcement. The table below provides the bearing capacity factor based on the reinforcement condition, tolerable rut depth, and traffic.

Table 19-28, Bearing Capacity Factors for USFS Method (adopted from Geosynthetics Engineering – August 2008)

Reinforcement	Tolerable Rut (inches)	Traffic (18 kip ESALs)	Bearing Capacity Factor (N_c)
Without	< 2	> 1,000	2.8
	2 to 4	100 – 1,000	3.0
	> 4	< 100	3.3
Geotextiles	< 2	> 1,000	5.0
	2 to 4	100 – 1,000	5.5
	> 4	< 100	6.0
Geogrids	< 2	> 1,000	5.8

Step six consists of determining the amount of bridge lift material required for both the unreinforced as well as the reinforced subgrade. The material thicknesses are determined from Figures 19-48 for single wheel loads and 19-49 for dual wheel loads depending on the vehicular configuration assumed in the third step.

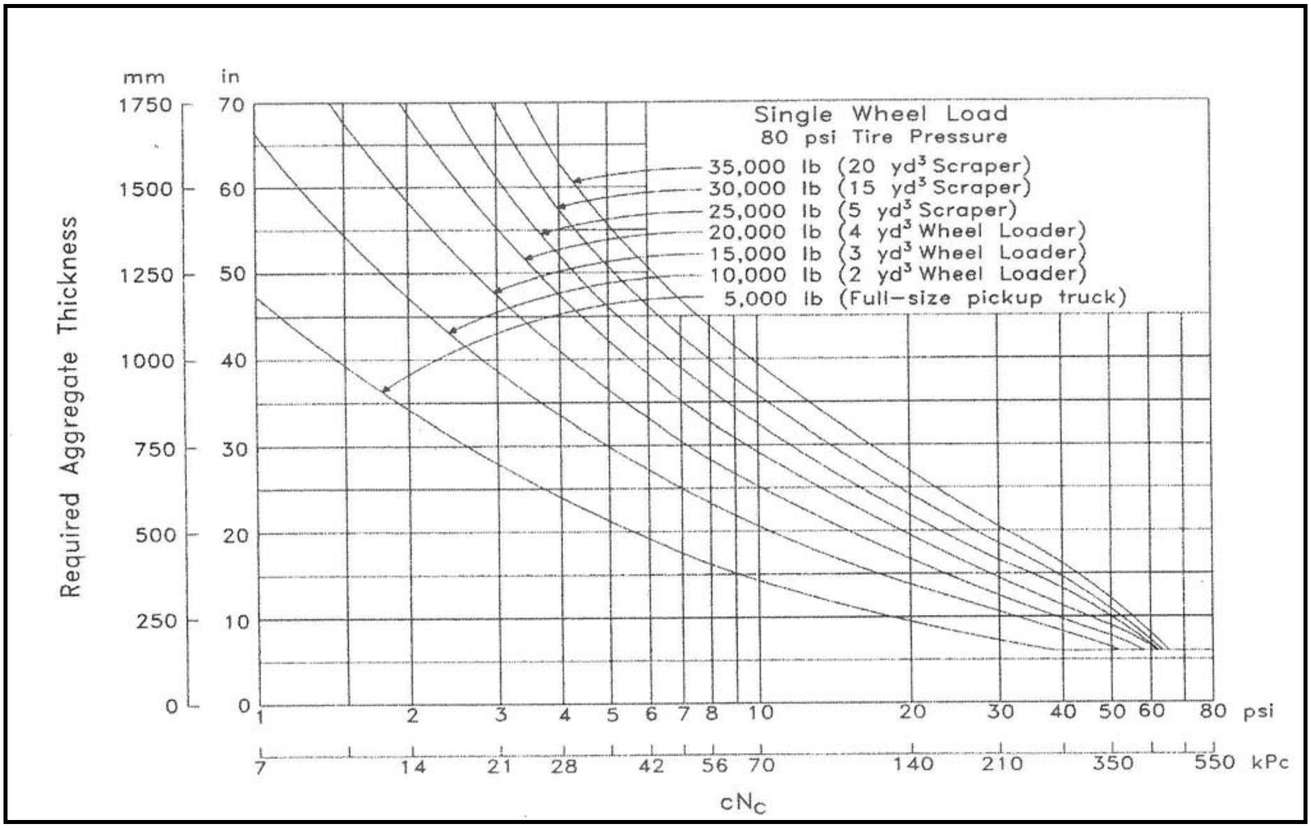

Figure 19-48, USFS Method Bridge Lift Thickness – Single Wheel Loads (adopted from Geosynthetics Engineering – August 2008)

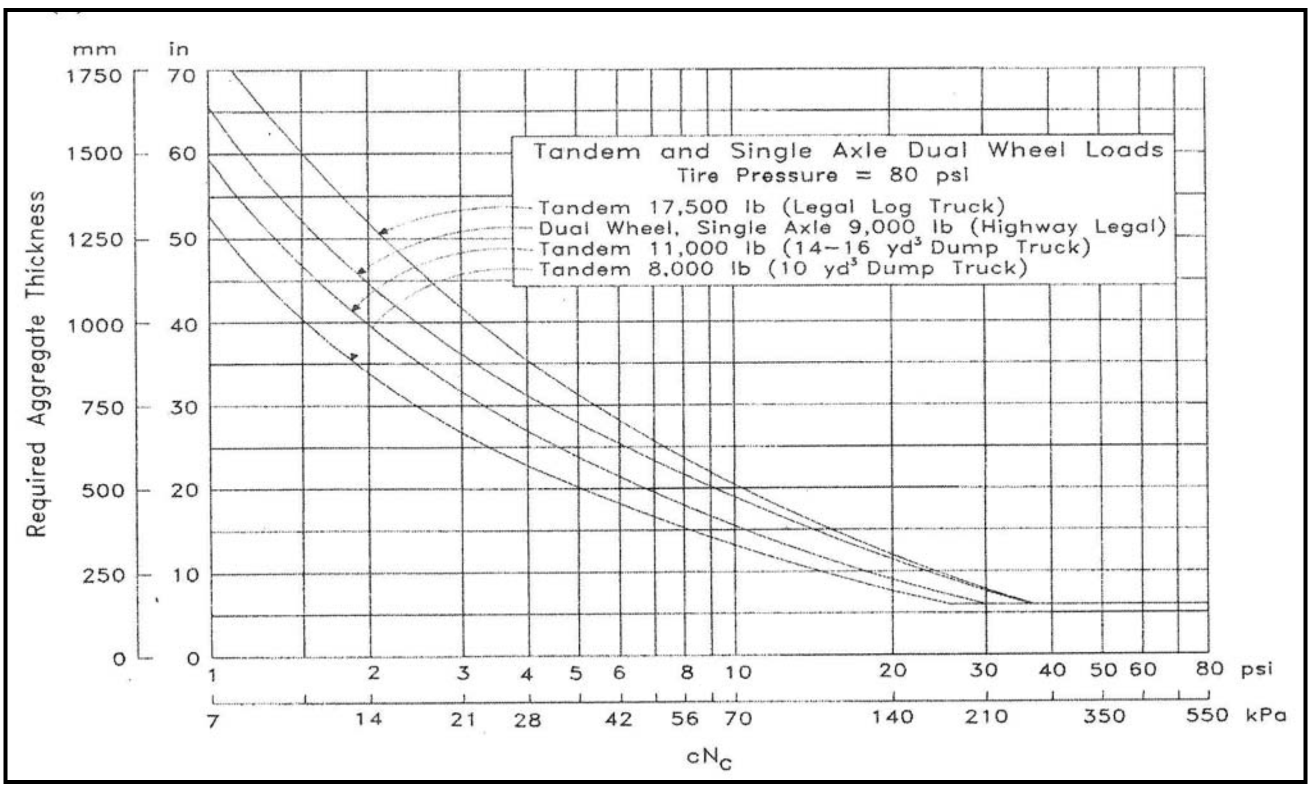

Figure 19-49, USFS Method Bridge Lift Thickness – Dual Wheel Loads (adopted from Geosynthetics Engineering – August 2008)

The seventh step is the selection on the design thickness of the bridge lift as well as material for the bridge lift. The thickness of the bridge lift should be rounded to the next higher thickness divisible by three. The USFS Method is also based on the bridge lift having an in-place CBR of 80, while the stone will obtain this CBR with little effort, the use of granular backfill, having a CBR much lower than 80, requires that the thickness of the bridge lift be increased. Increase the thickness of the bridge lift 3 inches for the use of granular bridge lift materials.

The eighth step is determine the survivability of the geotextile materials for the given soil conditions. Given the anticipated conditions that bridge lifts and reinforcement will be used on, a high survivability is required.

The final step in the USFS Method is developing any plan notes required.

19.10.1.2 Giroud- Han Method

The Giroud-Han method is an iterative process since the required thickness of bridging material is on both sides of the following equation:

$$\boldsymbol{h} = \frac{0.868 + 4\left(0.661 - 1.006J^2\right)\left(\frac{6.3}{h}\right)^{1.5}}{1.816}\left(\sqrt{\frac{80}{\frac{s}{3}\left(1 - 0.9e^{-\left(\frac{6.3}{h}\right)^2}\right)N_c c_u}} - 1\right)6.3 \quad \text{Equation 19-57}$$

Where, $$\left(0.661 - 1.006J^2\right) > 0 \quad \text{Equation 19-58}$$

Where,

h = Required bridge lift thickness (inches)
J = Aperture stability modulus
τ = c_u = Undrained shear strength (psi)
s = Maximum rut depth (inches)
N_c = Bearing Capacity Factor (see Table 19-30)

Table 19-29, Bearing Capacity Factor and Aperture Stability Modulus (adopted from Geosynthetics Engineering – August 2008)

	Bearing Capacity Factor (N_c)	**Aperture Stability Modulus ($J^{1,2}$)**
Unreinforced	3.14	0
Geotextile Reinforced	5.17	0
Geogrid Reinforced	5.71	$J^{1,2}$

[1]Aperture Stability Modulus determined by geogrid manufacturer/supplier

[2](dimensionless in Equation 19-56, but reported in N-m/degree)

The following assumptions and limitations are placed on the Grioud-Han method.

- Rut depth (s) is limited to 2 to 4 inches
- CBR of stone is greater than 30
- CBR of granular material (A-1 through A-2-6) is greater than 10
- The number Equivalent Single Axle Loads (ESALs) is 10,000
- The tension membrane effect was not taken into account, since it is negligible for rut depths less than 4 inches
- The radius of tire contact (r) is 6.3 inches
- Tire pressure (p) is 80 pounds per square inch
- The wheel load (P) is 10.0 kips
- The minimum thickness of bridge lift is 6 inches

The capacity of the existing subgrade soils should be determined to check whether reinforcement is needed or not using the following equation.

$$P_{h=0,unreinf} = \left(\frac{s}{3}\right) 391.53 c_u \qquad \text{Equation 19-59}$$

Where,

$P_{h=0,\ unreinf}$ = Unreinforced subgrade support capacity with no bridge lift

If $P_{h=0,\ unreinf}$ is greater than P, no reinforcement is required; however, a 6-inch bridge lift is recommended to prevent disturbance of the existing subgrade. If P is greater than $P_{h=0,\ unreinf}$, then reinforcement is required and Equation 19-56 should be used to determine the required thickness of bridge lift. Utilizing a P of 10,000 pounds in Equation 19-58, the minimum undrained shear strength with corresponding rut depth is shown in the following table.

Table 19-30, Minimum Undrained Shear Strength versus Rut Depth

Rut Depth (inches)	**Undrained Shear Strength (c_u, τ) (psf)**
2	5520
3	3675
4	2750

19.10.2 Reinforced Embankment

The design approach for the reinforced embankment is to prevent failure. Figure 19-50 provides depictions of potential modes of failure. These potential modes of failure indicate the type of analyses that will be required. In addition, the settlement of the embankment and the creep rate of the reinforcement needs to be considered as well. Creep of the reinforcement only becomes an issue if the creep rate of the reinforcement is greater than the increase in shear strength of the subgrade soils. The most critical condition for embankment stability is at the end of construction. Therefore, a total stress analysis should be performed. A reinforced embankment is different from a Reinforced Soil Slope (RSS – Chapter 18 and Appendix D). An RSS is defined in Chapter 18 as having slopes ranging from 2H:1V to 1H:1V. A reinforced embankment has flatter slopes and up to three layers of reinforcement at the bottom of the embankment.

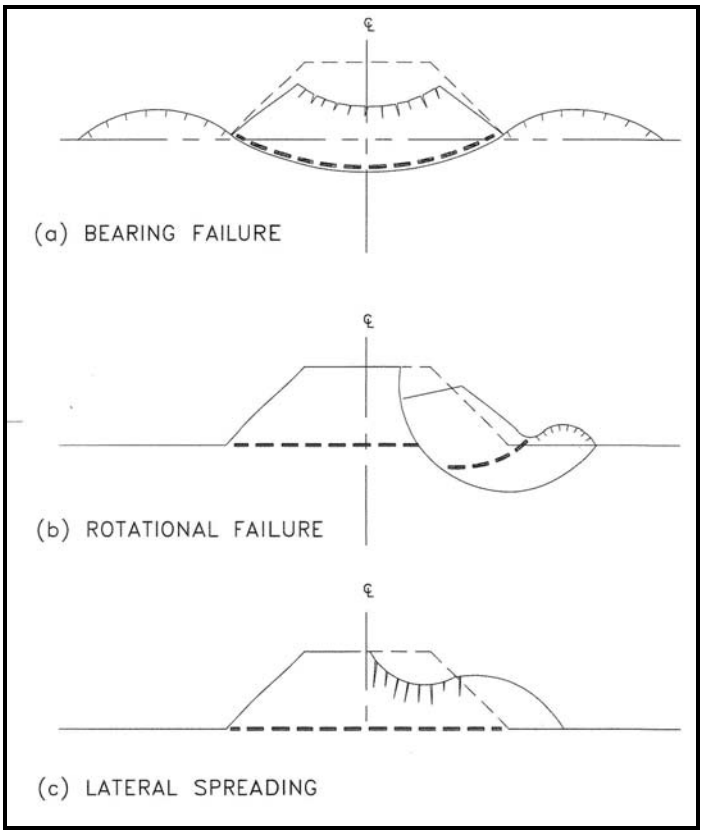

Figure 19-50, Reinforced Embankment Failure Modes (Ground Improvement Methods – August 2006)

19.10.2.1 Reinforced Embankment Design

The following design procedure is adopted from the design steps presented in the Ground Improvement Methods manual.

1. Geometry and Loading Conditions – The geometric parameters required are the height and length of the embankment over the soft foundation soils, the width of the crest (shoulder break-to-shoulder break), and the slide slope angle. The loading conditions include any surcharges and any temporary or dynamic loads. The construction rate

should also be included, because the gain in shear strength is directly affected by the placement of the embankments.

2. Soil Profile and Engineering Properties – The subsurface stratigraphy should be determined, including soil layering and groundwater table location for the foundation soils. The testing should include basic classification testing. The shear strength and consolidation properties should be determined either from correlations with field testing or from laboratory testing. The spatial variation (length and depth) of the soil properties should also be determined.

3. Embankment Fill Engineering Properties – The engineering properties of the fill material should be determined, including basic classification testing (grain-size distribution and moisture-plasticity relationship), moisture-density relationship, shear strength, and chemical properties. The first 18 to 24 inches of fill materials shall consist of free draining granular materials. Above this material, normal backfill materials may be placed.

4. Establish Resistance Factors and Performance Limits – The Resistance Factors and Performance Limits shall meet the requirements contained in Chapters 9 and 10, respectively. The stability analyses performed in the following steps are for the Strength limit state at the end of construction. The end of construction is the most critical condition. Therefore, the Extreme Event will not be checked. The Extreme Event will be checked using the shear strength anticipated from the increase with time (see Chapter 17).

5. Bearing Capacity Check – When the thickness of the soft foundation soil is much greater than the width of the embankment, the following equation may be used to determine the ultimate bearing capacity:

$$q_{ult} = \gamma_{fill} H = c_u N_c \qquad \text{Equation 19-60}$$

   Where,

   q_{ult} = Ultimate bearing capacity
   N_c = 5.14
   c_u = Undrained shear strength of foundation soil
   γ_{fill} = Unit weight of fill material
   H = Height of embankment

   If the thickness of the soft soil is less than the width of the embankment, check lateral squeeze using the procedure provided in Chapter 17.

6. Rotational Shear Stability Check – Perform a rotational slip surface analysis (see Chapter 17) on the unreinforced embankment and foundation to determine the critical failure surface and the resistance factor against local shear instability. If the calculated resistance factor is less than required, then, reinforcement is not required. If the resistance factor is greater than required, then, calculate the required reinforcement strength (T_g) to provide an adequate resistance factor using the Figure 19-51 and the following equation:

$$T_g = \frac{\Delta M_R}{y}$$ Equation 19-61

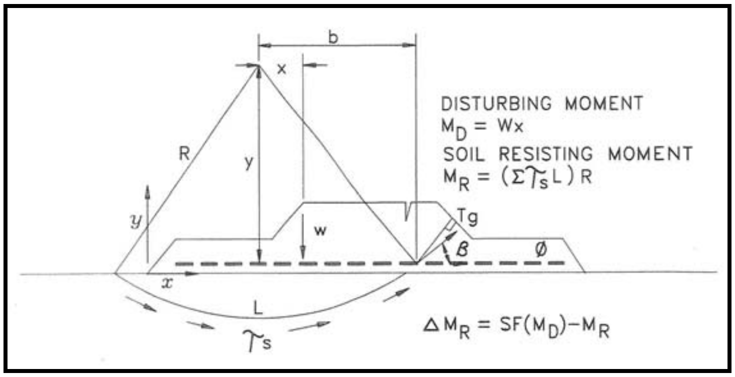

Figure 19-51, Rotational Failure Model
(Ground Improvement Methods – August 2006)

Note: SF = 1/φ

7. Sliding Block Stability Check – Perform a sliding block analysis (see Chapter 17). If the calculated resistance factor is less than required, then, reinforcement is not required. If the resistance factor is inadequate, then, determine the lateral spreading strength of reinforcement (T_{ls}) required (Figure 19-52). The soil/geosynthetic cohesion, C_a, should be assumed to be 0 for extremely soft soils and low embankments. A cohesion value should be included with placement of all subsequent fills in staged embankment construction. In addition to checking for rupture, sliding of the embankment, the sliding of the embankment on top the reinforcement, should be checked (Figure 19-53).

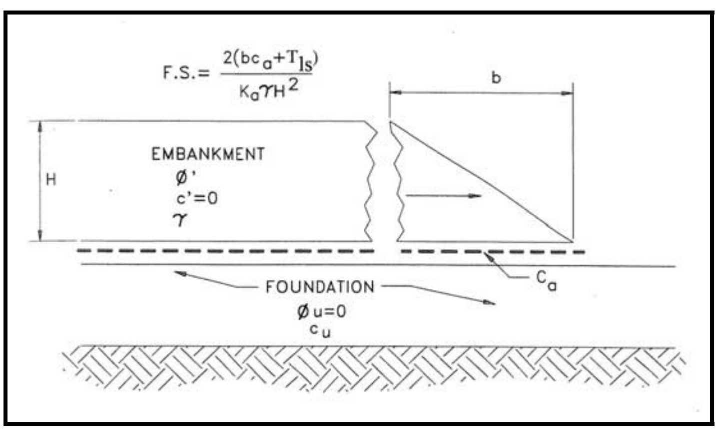

Figure 19-52, Sliding Failure – Rupture of Reinforcement
(Ground Improvement Methods – August 2006)

Note: FS = 1/φ

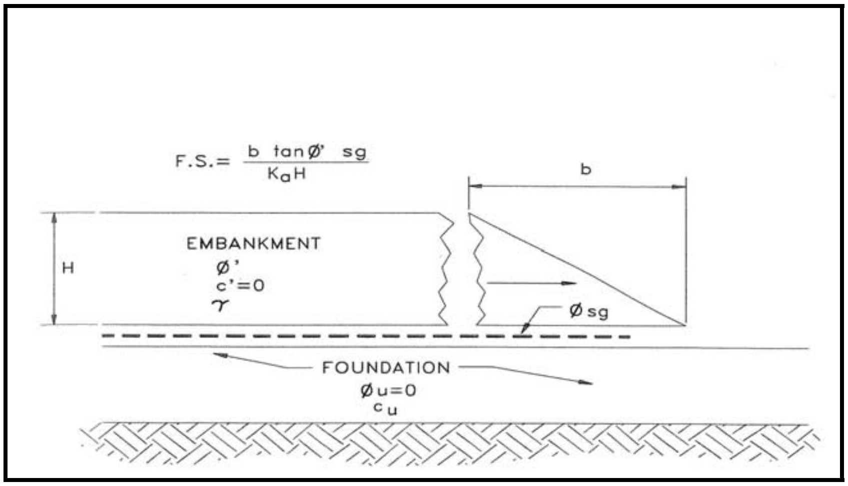

Figure 19-53, Sliding Failure of Embankment over Reinforcement (Ground Improvement Methods – August 2006)

Note: FS = 1/φ

8. Establish Tolerable Geosynthetic Deformation - The deformation of the geosynthetic reinforcement is required to develop the tensile capacity required to prevent failure. The strain in the geosynthetic reinforcement is provided in the following equation.

$$\varepsilon_{geosyn} = \frac{T_{ls}}{J} \qquad \text{Equation 19-62}$$

Where,

ε_{geosyn} = Strain in the geosynthetic

Cohesionless Soils: ε_{geosyn} = 5 to 10 percent

Cohesive Soils: ε_{geosyn} = 2 percent

Peats: ε_{geosyn} = 2 to 10 percent

T_{ls} = Lateral spreading strength of reinforcement

J = Reinforcement Modulus

The maximum strain in the geosynthetic reinforcement will be approximately twice the average strain in the embankment.

9. Establish Geosynthetic Strength Requirements – Most embankments are relatively long and narrow in shape. Thus, during construction, stresses are imposed on the geosynthetic in the longitudinal direction (i.e., along the direction of the centerline). Reinforcement may also be required for loadings that occur at bridge abutments, and due to differential settlements and embankment bending, especially over nonuniform foundation conditions and at the edges of soft soil deposits, because both rotational and sliding block failures are possible in the direction along the alignment of the embankment. This determines the longitudinal strength requirements of the geosynthetic. Because the usual placement of the geosynthetic is in strips perpendicular to the centerline, the longitudinal stability will be controlled by the strength of the transverse seams.

10. Selection of Geosynthetic Reinforcement – Once the geosynthetic strength requirements are established, the geosynthetic reinforcement should be selected that meets the required strength and deformation (strain) requirements.

11. Estimate Magnitude and Rate of Embankment Settlement – The magnitude and rate of embankment settlement should be determined using the procedures outlined in Chapter 17.

12. Establish Construction Sequence and Procedures – The construction sequence and procedures should be established. Proper placement and performance of the geosynthetic is highly influenced by the construction sequence and procedure. The sequence and procedure should be as clear and concise as possible to prevent misunderstandings during construction.

13. Establish Construction Observation Requirements – Since implemented construction procedures are crucial to the success of reinforced embankments on very soft foundations, competent and professional construction inspection is absolutely essential. Field personnel must be properly trained to observe every phase of the construction and to ensure that:

    a. The specified material is delivered to the project
    b. The geosynthetic is not damaged during construction
    c. The specified construction sequence is explicitly followed

    Instrumentation requirements should also be established. As a minimum, install piezometers, settlement points, surface survey points and slope inclinometers. Part of the instrumentation requirements is establishing who will obtain the measurements and how often the measurements will be obtained.

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Chapter 20

GEOSYNTHETIC DESIGN

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

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CHAPTER 20

GEOSYNTHETIC DESIGN

20.1 INTRODUCTION

According to Geosynthetics Engineering, FHWA NHI-07-092, August 2008, "ASTM (2006) D 4439 defines a geosynthetic as a planar product manufactured from a polymeric material used with soil, rock, earth or other geotechnical-related material as an integral part of a civil engineering project, structure, or system." Geosynthetic materials are comprised of four basic groups: geotextiles, geogrids, geomembranes, and geocomposites. Each of these basic groups is discussed in greater detail below. The materials making up the geosynthetics may be either man-made or natural. The most common man-made materials consist of synthetic polymers such as polypropylene, polyester and polyethylene. It should be noted that these polymers are highly resistant to biological and chemical degradation. Other man-made polymers used less frequently are polyamides, polyvinyl chloride (PVC) and glass fibers. The polyamides (e.g., nylon) are not very durable in the soil since they tend to soften in the presence of water. Natural materials consist of cotton, jute, etc. However, natural materials are biodegradable and, therefore, should be used for temporary (< 1 year) applications only. Figure 20-1 provides a classification scheme for geosynthetics. Selected geosynthetic terms are provided in Section 20.9. These terms are based on ASTM (2006) D 4439 *Standard Terminology for Geosynthetics*.

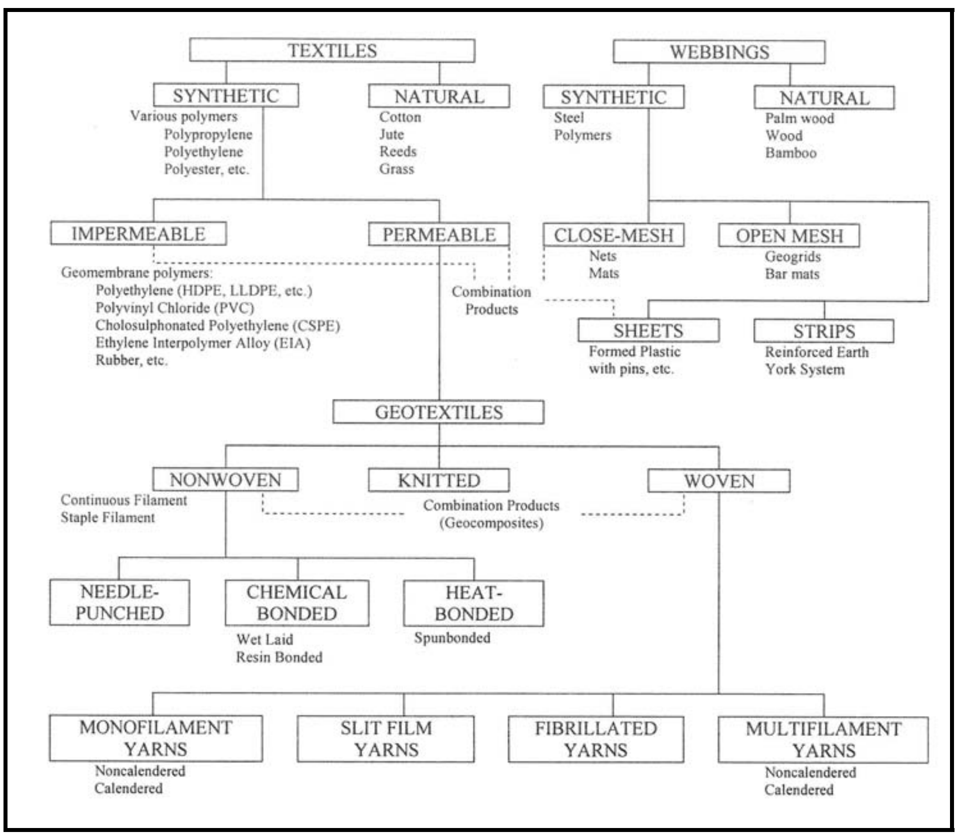

Figure 20-1, Geosynthetic Classification Scheme (Geosynthetics Engineering – August 2008)

Geosynthetics are identified using the information contained in Table 20-1.

Table 20-1, Geosynthetic Generic Identifiers
(adopted from Geosynthetics Engineering – August 2008)

Identifier	Descriptive Term	Example
Polymer	High density, low density, etc.	**High density polyethylene** geomembrane
Type of Element	Filament, yarn, type, strand, rib, coated rib, etc.	Polypropylene staple **filament** needlepunched nonwoven geotextile, 10 oz/yd^2
Manufacturing Process	Woven, needlepunched nonwoven, heatbonded nonwoven, stitchbonded, extruded, knitted, welded, uniaxial, biaxial, roughened sheet, smooth sheet, etc.	Polypropylene staple filament **needlepunched nonwoven** geotextile, 10 oz/yd^2
Primary Geosynthetic Type	Geotextile, geogrid, geomembrane, etc.	High density polyethylene **geomembrane**
Mass per Unit Area or Thickness[1]	Mass per unit area – geotextiles Thickness - geomembranes	Polypropylene staple filament needlepunched nonwoven geotextile, **10 oz/yd^2**
Additional Information	As required	Polypropylene extruded biaxial geogrid, **with 1 in x 1 in openings**

[1]As appropriate

20.2 GEOTEXTILES

Geotextiles are subdivided into two categories, woven and nonwoven. Both categories are comprised of fibers or yarns that are combined into a planar textile structure. The fibers can be either continuous filaments or staple fibers. The continuous filaments are very long thin polymeric strands, whereas, the staple fibers are short (3/4 to 6 inches) filaments. Both filament types can be manufactured from an extruded plastic sheet that is slit to form thin, flat tapes. The extrusion or drawing process elongates the polymers in the direction of the draw causing an increase in the strength of the filament. After the drawing process, filaments may also be fibrillated, a process in which the filaments are split into finer filaments by crimping, twisting, cutting, or nipping with a pinned roller. Fibrillation provides pliable, multifilament yarns with a more open structure that are easier to weave.

Woven geotextiles are made of monofilament, multifilament, or fibrillated yarns or of slit film tapes. This category of geotextiles is manufactured similarly to cloth or other textiles, using traditional weaving techniques. In nonwoven geotextile manufacturing, the polymeric fibers or filaments are continuously extruded and spun, blown or otherwise, placed onto a moving conveyor belt. The mass of fibers or filaments are then either needlepunched or heat bonded. Needlepunch is the process of mechanically entangling the fibers or filaments using a series of small needles. Heat bonding is the process in which individual fibers or filaments are welded together by heat and pressure at contact points to create a nonwoven mass.

20.3 GEOGRIDS

Geogrids are formed by regular network of tensile elements with openings of sufficient size to allow interlock with the surrounding fill materials. The primary purpose of geogrids is reinforcement. Geogrids can be manufactured in three ways; extrusion, knitting or weaving, and welding. Extruded geogrids, also called integral geogrids, are manufactured with integral junctions by extruding and orienting polymeric (polyethylene or polypropylene) materials. Geogrids may also be manufactured of multifilament polyester yarns, joined at the crossover points by a knitting or weaving processes and then encased with a polymer-based, plasticized coating. Welded geogrids are manufactured by welding polymeric strips together at the cross over points. All of these manufacturing techniques allow geogrids to be oriented such that the principal strength is in one direction (uniaxial geogrids) or in both directions (biaxial geogrids). In biaxial geogrids, the strength is not the same in both directions.

20.4 GEOMEMBRANES

Geomembranes, unlike geotextiles and geogrids, are manufactured with a single, solid sheet of geosynthetic material. Geomembranes are used as either a low-permeability or impermeable boundary to prevent the movement of fluids (either liquid or gas). The primary use of geomembranes is in the design and construction of landfills; however, geomembranes have selected uses on transportation projects, such as being used as an impermeable barrier above structural backfill behind an ERS or above lightweight EPS materials (see Figure 20-2). Because there are limited requirements for the use of geomembranes in transportation projects, geomembranes will not be discussed in the remainder of this Chapter.

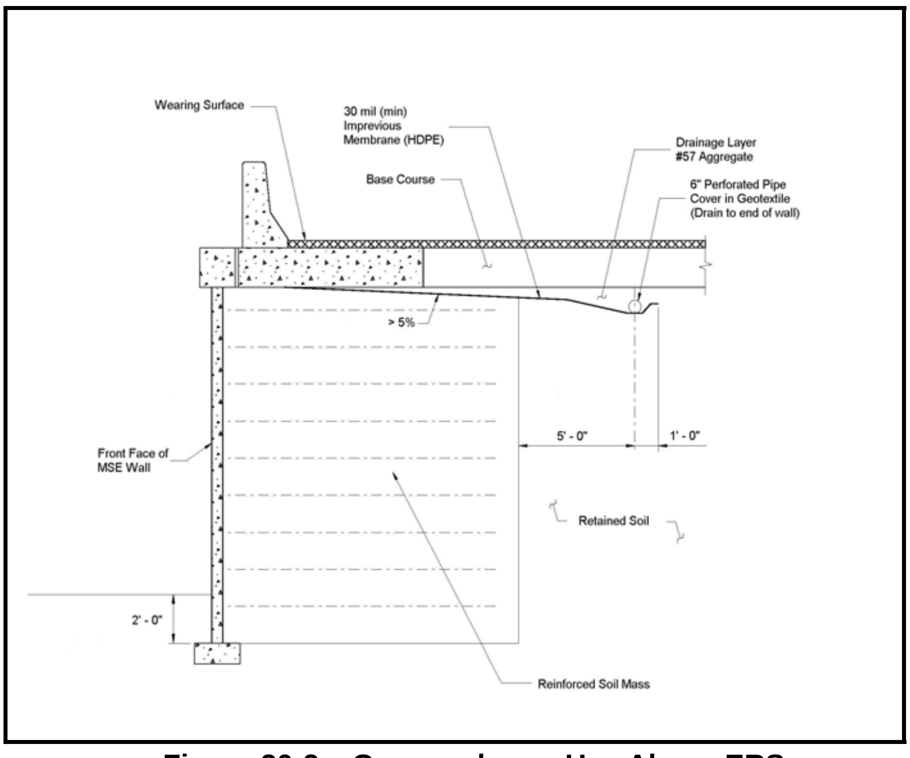

Figure 20-2, Geomembrane Use Above ERS
(Earth Retaining Structures – June 2008)

20.5 GEOCOMPOSITES

Geocomposites are the combination of two or more geosynthetic materials combined together, such as geotextiles and geogrids. Most geocomposites are used in either drainage applications or waste-containment. Prefabricated vertical drains (PVDs) are an example of geocomposites. Included in geocomposites are the three-dimensional polymeric cell structures.

20.6 FUNCTIONS AND APPLICATIONS

Geosynthetics have six primary functions as listed below:

- Filtration
- Drainage
- Separation
- Reinforcement
- Fluid Barrier
- Protection

20.6.1 Filtration

Geotextiles (woven and nonwoven) are used as filters to prevent soils from migrating into drainage aggregate or pipes, while maintaining water flow through the system. These materials are also used below riprap and other armor materials to prevent erosion of the soils from the stream bank. For a geotextile to function as a filter, the Apparent Opening Size (AOS) must be smaller than the smallest size particle to be retained and still allow for the flow of water through the geotextile material. To provide good filtration, a geotextile should meet the criteria provided in the following equation.

$$\boldsymbol{AOS \leq BD_{85(soil)}} \qquad \text{Equation 20-1}$$

Where,

AOS = Apparent Opening Size (see Section 20.9 - Geosynthetic Terminology)
$D_{85(soil)}$ = Diameter of soil particle for which 85 percent are smaller
B = Dimensionless coefficient rated to C_u

For granular soils B, for both woven and nonwoven geotextiles, is determined from the following equations.

$$\boldsymbol{B = 1 \qquad C_u \leq 2 \quad \text{or} \quad C_u \geq 8} \qquad \text{Equation 20-2}$$

$$\boldsymbol{B = 0.5C_u \qquad 2 \leq C_u \leq 4} \qquad \text{Equation 20-3}$$

$$\boldsymbol{B = \frac{8}{C_u} \qquad 4 < C_u < 8} \qquad \text{Equation 20-4}$$

For fine-grained soils, B is a function of the type of geotextile.

Woven geotextiles

$$B = 1 \qquad \text{Equation 20-5}$$

Nonwoven geotextiles

$$B = 1.8 \qquad \text{Equation 20-6}$$

In addition to the AOS, the permeability and permittivity of the geotextile requires consideration. The selection of the correct filter is based on the critical/severe nature of the project. The criteria for critical/severe projects are provided in the following table.

Table 20-2, Guidelines for Evaluating Critical/Severe Nature (adopted from Geosynthetics Engineering – August 2008)

A. Critical Nature of Project		
Item	**Critical**	**Less Critical**
Risk of loss of life and/or structural damage due to drain failure	High	None
Repair cost versus installation costs of drain	>>>	= or <
Evidence of drain clogging before potential catastrophic failure	None	Yes
B. Severity of Conditions		
Item	**Severe**	**Less Severe**
Soil to be drained	Gap-graded, pipable, or dispersible	Well-graded or uniform
Hydraulic gradient	High	Low
Flow conditions	Dynamic, cyclic, or pulsating	Steady state

For less critical applications and less severe conditions,

$$k_{geotextile} \geq k_{soil} \qquad \text{Equation 20-7}$$

For critical applications and severe conditions,

$$k_{geotextile} \geq 10 k_{soil} \qquad \text{Equation 20-8}$$

Where,

$k_{geotextile}$ = Coefficient of permeability of geotextile
k_{soi} = Coefficient of permeability of soil

Permittivity is the coefficient of permeability normal to the plane of the geotextile divided by the thickness of the geotextile material (see Figure 20-3)

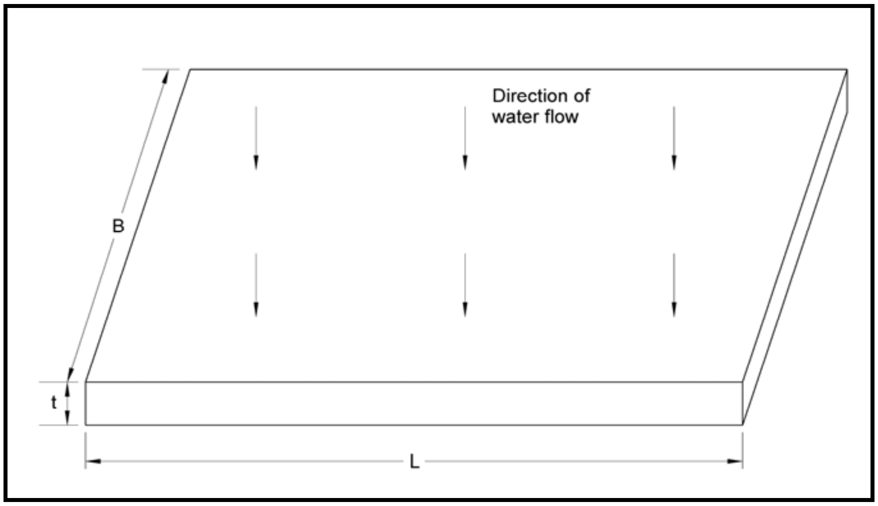

Figure 20-3, Permittivity
(modified from Shulka – 2002)

$$Q_n = \Psi \Delta h A_n \qquad \text{Equation 20-9}$$

$$\Psi = \frac{k_n}{t} \qquad \text{Equation 20-10}$$

$$A_n = LB \qquad \text{Equation 20-11}$$

Where,

Q_n = Normal volumetric flow
Δh = Head causing flow
k_n= Coefficient of permeability normal to geotextile
t = Thickness of geotextile
Ψ = Permittivity of geotextile
L = Length of geotextile
B = Width of geotextile

The permittivity requirements depend on the fines content of the soil to be filtered. The more fines in the soil, the greater the permittivity required. The following table contains approximate fines content and recommended permittivity requirements based on previous experience.

Table 20-3, Permittivity Requirements
(adopted from Geosynthetics Engineering – August 2008)

Percent Passing No. 200 Sieve	Ψ (sec^{-1})
< 15	≥ 0.5
15 – 50	≥ 0.2
> 50	≥ 0.1

20.6.2 Drainage

Nonwoven needlepunched geotextiles and geocomposites can also provide drainage by allowing water to drain from or through low permeability soils. The primary application of the use of geotextiles for drainage is in dissipation of excess pore pressures. In some cases, the nonwoven geotextile is thick and will allow the flow of water through geotextile material itself. This flow of water through the geotextile material is called transmissivity. Transmissivity is the product of the permeability of the geotextile for in plane water flow and the thickness of the geotextile (see figure below).

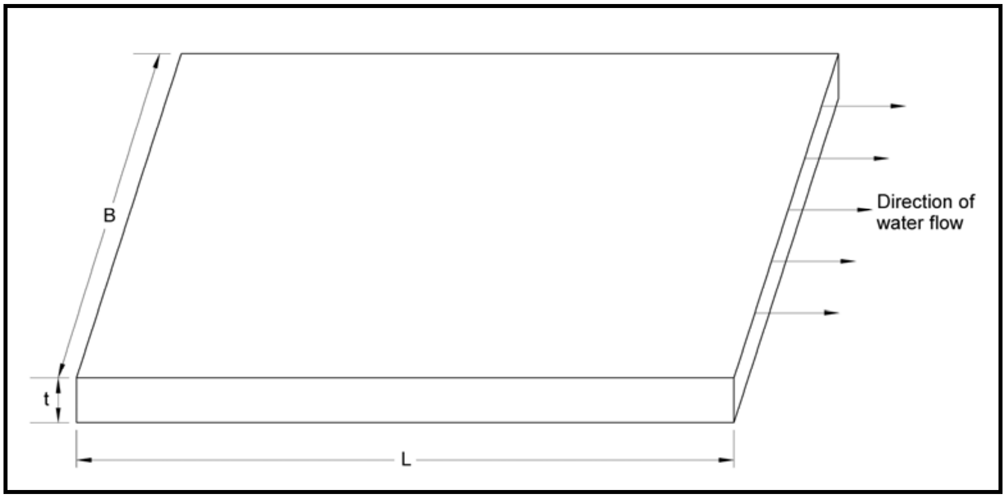

Figure 20-4, Transmissivity (modified from Shulka – 2002)

$$\boldsymbol{Q_p} = \Theta \boldsymbol{iB} \qquad \text{Equation 20-12}$$

$$\Theta = \boldsymbol{k_p t} \qquad \text{Equation 20-13}$$

$$\boldsymbol{i} = \frac{\Delta \boldsymbol{h}}{\boldsymbol{L}} \qquad \text{Equation 20-14}$$

Where,

Q_p = In-plane volumetric flow
Δh = Head causing flow
k_p= Coefficient of permeability in plane to geotextile
t = Thickness of geotextile
Θ = Transmissivity of geotextile
L = Length of geotextile
B = Width of geotextile

20.6.3 Separators

Geosynthetic materials, primarily geotextiles, are used to prevent the migration of fines from subgrade soils into granular bases. The AOS of the geotextile should be sized to prevent the migration of fines. In addition, geogrids may also be used as a separator to prevent the migration of granular materials (aggregate) into fine-grained, soft subgrade soils (see Figure 20-5). However, this application will not prevent the migration of fines from the subgrade soil into the aggregate.

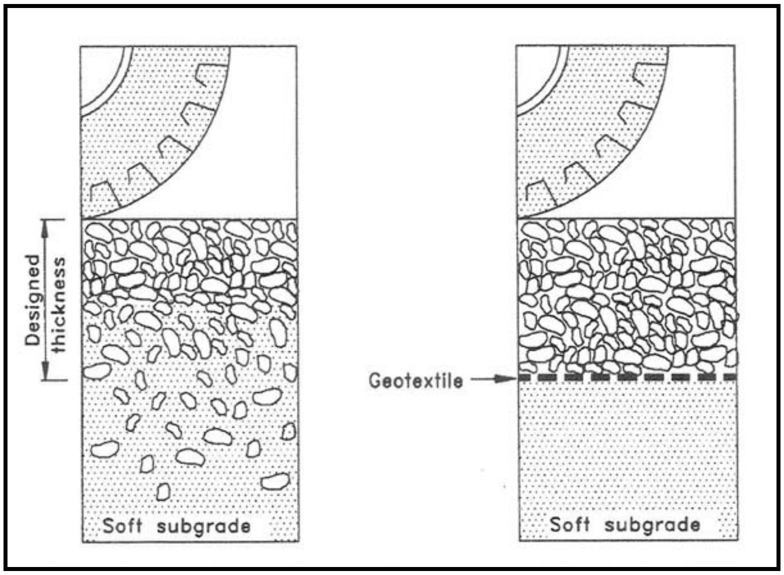

Figure 20-5, Geotextile Soil Separator
(Geosynthetics Engineering – August 2008)

20.6.4 Reinforcement

Both geotextiles and geogrids are used as reinforcement. These materials add tensile strength to a soil matrix, thus providing a more competent and stable material. Reinforcement enables embankments to be constructed over very soft foundation soils (see Chapter 19) and permits the construction of steep slopes and ERSs (see Chapter 18 and Appendices C and D).

20.6.5 Fluid Barriers

Geomembranes, geotextile composites, and geosynthetic clay liners are used as fluid barriers to impede the flow of a liquid or a gas from one location to another. As indicated previously, fluid barriers (geomembranes) are used in lightweight (EPS) fill applications for transportation projects. Fluid barriers have limited applications in transportation projects; therefore, there will be no detailed discussion of fluid barriers. If a fluid barrier is required, please refer to Geosynthetics Engineering, FHWA NHI-07-092, dated August 2008 for additional details.

20.6.6 Protection

Geosynthetics provide protection by providing a stress relief layer. Temporary geosynthetic blankets and permanent geosynthetic mats are placed over the soil to reduce erosion caused by rainfall impact and water flow shear stress.

20.6.7 Secondary Applications

The previous sections indicate the various primary functions of geosynthetics; however, geosynthetics can provide several secondary functions while providing the primary function. Provided in the following table are the primary function and some of the secondary functions.

Table 20-4, Primary and Secondary Functions
(adopted from Geosynthetics Engineering – August 2008)

Primary Function	Secondary Function	Selected Applications[1]
Filtration	Separation, drainage, protection, reinforcement	Trench drains, pipe wrapping, base course drains
Drainage	Separation, filtration	Retaining walls, vertical and horizontal drains
Separation	Filtration, drainage, reinforcement	Working platforms, embankment construction
Reinforcement	Filtration, drainage, separation	Base reinforcement, fill reinforcement, load redistribution
Fluid Barrier	Protection	Asphalt overlays[2]
Protection	Reinforcement, fluid barrier	Permanent and temporary erosion control

[1]More applications are possible; this is not a complete list

[2]Not currently allowed by SCDOT, presented here only for information

20.7 DESIGN APPROACH

Geosynthetics Engineering recommends the following design procedure for geosynthetics:

1. "Define the purpose and establish the scope of the project.
2. Investigate and establish the geotechnical conditions at the site (geology, subsurface exploration, laboratory and field testing, etc.)
3. Establish application criticality, severity, and performance criteria. Determine external factors that may influence geosynthetic performance.
4. Formulate trial designs and compare several alternatives.
5. Establish the models to be analyzed, determine the parameters, and carry out the analysis.
6. Compare results and select the most appropriate design; consider alternatives versus cost, construction feasibility, etc. Modify design if necessary.
7. Prepare detailed plans and specifications including: a) specific property requirements for the geosynthetic; and b) detailed installation procedures.
8. Hold preconstruction meeting with contractor and inspectors.
9. Approve geosynthetic on the basis of specimens' laboratory test results and/or manufacturer's certification.
10. Monitor construction.
11. Inspect after major events (e.g., 100-year rainfall or an earthquake) that may compromise system performance."

20.8 EVALUATION OF PROPERTIES

The required geosynthetic design properties depend on the specific application and associated function the geosynthetic is to provide. Table 20-5 provides a list of properties that cover the range of important criteria that are required to evaluate a geosynthetic for most applications. It should be noted that not all of the listed requirements will be necessary for all applications. Typically six to eight properties are required for a specific application. The properties required for mechanical or hydraulic design are different from those required for constructability (survivability) and durability. Table 20-6 provides typical SCDOT applications along with the associated function. Tables 20-4, 20-5 and 20-6 should be used in conjunction to determine the appropriate properties for each application. All geosynthetic material properties can be placed into three basic categories: general, index, and performance. General properties are usually provided by the manufacturer or distributor or from publically available literature. Index properties were originally developed by manufacturers for quality control purposes and only provide an indication or qualitative assessment of the property of interest. Performance tests are an attempt to model the soil-geosynthetic interaction and, therefore, require the geosynthetic to be tested together with on site soils. This type of testing provides a direct measure of specific properties of interest.

Table 20-5, Important Criteria and Principal Properties (adopted from Geosynthetics Engineering – August 2008)

Criteria and Parameter	Property[1]	Function					
		Filtration	Drainage	Separation	Reinforcement	Fluid Barrier	Protection
Design Requirements:							
Mechanical Strength							
Tensile Strength	Wide Width Strength	-	-	-	✓	✓	-
Tensile Modulus	Wide Width Modulus	-	-	-	✓	✓	-
Seam Strength	Wide Width Strength	-	-	-	✓	✓	-
Tension Creep	Creep Resistance	-	-	-	✓	✓	-
Compression Creep	Creep Resistance	-	✓[2]	-	-	-	-
Soil-Geosynthetic Friction	Shear Strength	-	-	-	✓	✓	✓
Hydraulic							
Flow Capacity	Permeability	✓	✓	✓	✓	✓	-
	Transmissivity	-	✓	-	-	-	✓
Piping Resistance	AOS	✓	-	✓	✓	-	✓
	Porimetry	✓	-	-	-	-	✓
Clogging Resistance	Gradient Ratio or Long-Term Flow	✓	-	-	-	-	✓
Constructability Requirements:							
Tensile Strength	Grab Strength	✓	✓	✓	✓	✓	✓
Seam Strength	Grab Strength	✓	✓	✓	-	✓	-
Bursting Resistance	Burst Strength	✓	✓	✓	✓	✓	✓
Puncture Resistance	Rod or Pyramid Puncture	✓	✓	✓	✓	✓	✓
Tear Resistance	Trapezoidal Tear	✓	✓	✓	✓	✓	✓
Durability:							
Abrasion Resistance[3]	Reciprocating Block Abrasion	✓	-	-	-	-	-
UV Stability[4]	UV Resistance	✓	-	-	✓	✓	✓
Soil Environment[5]	Chemical	✓	✓	?	✓	✓	?
	Biological	✓	✓	?	✓	✓	?
	Wet-Dry	✓	✓	-	-	-	✓
	Freeze-Thaw	✓	✓	-	-	✓	-

Notes:
1. See Table 1-4 of Geosynthetics Engineering, August 2008 for specific procedures.
2. Compression creep is applicable to some geocomposites.
3. Erosion control applications where armor stone may move.
4. Exposed geosynthetics only.
5. Where required.

Table 20-6, Evaluation of Geosynthetic Property Requirements (adopted from Geosynthetics Engineering – August 2008)

Application	Function			
	Filtration	Drainage	Separation	Reinforcement
Subsurface Drainage	☑		✓	
Prefabricated Drains	✓	☑	✓	
Hard Armor Erosion Control			✓	
Silt Fence			✓	
Subgrade Separation	✓		☑	
Subgrade Stabilization	☑	✓	☑	✓
Base/Subbase Reinforcement			✓	☑
Embankments over Soft Subgrade	☑	✓	✓	☑
Reinforced Slopes		✓		☑
Reinforced Soil Walls	☑			
✓ indicates Primary Function ☑ indicates Secondary Function				

20.9 GEOSYNTHETIC TERMINOLOGY

The geosynthetic terminology provided below is from Geosynthetic Engineering,

apparent opening size (AOS, O_{95}) – a property which indicates the approximate largest particle that would effectively pass through a geotextile

blinding – condition whereby soil particles block the surface openings of a geotextile, thereby reducing the hydraulic conductivity

California Bearing Ratio (CBR) – the ratio of (1) the force per unit area required to penetrate a soil mass with a 3-square-inch circular piston (approximately 2-inch diameter) at the rate of 0.05 inches/minute to (2) the force per unit area required for corresponding penetration of a standard method

clogging – condition where soil particles move into and are retained in the openings of a geotextile, thereby reducing hydraulic conductivity

cross-machine direction – the direction in the plane of the geosynthetic perpendicular to the direction of manufacture

filtration – the process of retaining soils while allowing the passage of water (fluid)

geocell – a three-dimensional comb-like structure, to be filled with soil, aggregate or concrete

geocomposite – a geosynthetic material manufactured of two or more materials.

geogrid – a geosynthetic formed by a regular network of tensile elements and apertures, typically used for reinforcement applications

geomembrane – an essentially impermeable geosynthetic, typically used to control fluid migration

geonet – a geosynthetic consisting of integrally connected parallel sets of ribs overlying similar sets of ribs, for planar drainage of liquids or gases

geosynthetic – a planar product manufactured from polymeric material used with soil, aggregate, or other geotechnical engineering materials

geotextile – a permeable geosynthetic comprised solely of textiles

index test – a test procedure which may contain a known bias but which may be used to establish an order for a set of specimens with respect to the property of interest

machine direction – the direction in the plane of the geosynthetic parallel to the direction of manufacture

minimum average roll value (MARV) – a quality control tool used by geosynthetic manufacturers to establish and publish minimum (or maximum) property values

permeability – the rate of flow of a liquid under a differential pressure through a material

permittivity – the volumetric flow rate of water per unit cross sectional area per unit head under laminar flow conditions, in the normal direction *(see Figure 20-3)* through a geotextile"

transmissivity - the volumetric flow rate of water per unit cross sectional area per unit head under laminar flow conditions, in the in-plane direction (see Figure 20-4) through a geotextile

20.10 REFERENCES

ASTM, 2006, *Standard Terminology for Geosynthetics*, D 4439, ASTM International.

Holtz, R.D., Christopher, B.R., and Berg, R.R., 2008, Geosynthetic Design and Construction Guidelines, FHWA NHI-07-092.

Tanyu, B. F., Sabatini, P. J., and Berg, R. R. (2008), Earth Retaining Structures, FHWA-NHI-07-071, Department of Transportation, Federal Highway Administration, Washington D.C.

Shukla, S.K. 2002, Geosynthetics and Their Design, Thomas Telford.

Chapter 21

GEOTECHNICAL REPORTS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

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CHAPTER 21

GEOTECHNICAL REPORTS

21.1 INTRODUCTION

This Chapter presents the requirements for the preparation of geotechnical reports that will be used to design projects both by and for SCDOT. Geotechnical reports are prepared to convey information concerning subsurface conditions, foundations and construction considerations (plan notes) to the project designers. SCDOT uses four basic types of reports to convey geotechnical and site-specific seismic response information.

- Geotechnical Base Line Report
- Preliminary Geotechnical Engineering Report
- Final Geotechnical Engineering Report
- Site-Specific Seismic Response Analysis Report

The Geotechnical Base Line Report (GBLR) is primarily issued in conjunction with Design/Build (D/B) projects. Typically a bridge replacement project will have both Bridge and Roadway Geotechnical Engineering Reports (Preliminary and Final) prepared. However, some bridge replacement projects may not require a Roadway Geotechnical Engineering Report (RGER), while some road projects may not require a Bridge Geotechnical Engineering Report (BGER). The PCS/GDS should be contacted if there is a question concerning whether a report (Roadway or Bridge) is required. Typically, Preliminary Bridge and Roadway Geotechnical Engineering Reports (PBGER and PRGER, respectively) will be issued for the project prior to the Design Field Review (DFR). Final Bridge and Roadway Geotechnical Engineering Reports (BGER and RGER, respectively) are issued at the completion of final field operations and generally after the DFR. In some circumstances a Site-Specific Seismic Response Analysis Report (SSSRAR) is prepared to provide an in-depth discussion of the seismic response of a specific site. The following Sections discuss each of the types of reports used by SCDOT.

21.2 GEOTECHNICAL BASE LINE REPORT

The GBLR is used to provide limited (preliminary) geotechnical information on a D/B project, thus permitting the contractor to bid on the project with a certain degree of knowledge and acceptable risk. A GBLR does not provide any engineering interpretations or final engineering recommendations. The GBLR should be used in conjunction with project specific D/B criteria. The GBLR should contain at a minimum an introduction, project description, objective and scope of the geotechnical exploration. Any variations from the field testing procedures as described in this Manual should be discussed or mentioned. The narrative portion of this type of report is anticipated to be relatively short, with the Appendices of the report being large. The Appendices should at a minimum contain project and testing location plans, field exploration records (soil test boring logs, cone penetrometer and dilatometer records), and the results of all laboratory testing. Each field exploration record should contain the location of the testing and should correspond to the testing location plan. Any guides used to interpret the data should also be included. The laboratory testing results should indicate the location and depth of each sample clearly on the test result. Any deviations from the laboratory testing procedures described in this Manual should be indicated.

21.3 BRIDGE GEOTECHNICAL ENGINEERING REPORTS

The Bridge Geotechnical Engineering Reports provide geotechnical information related specifically to the design of bridge foundations. The contents of both preliminary (PBGER) and final (BGER) geotechnical engineering reports are described in the following sections.

21.3.1 Preliminary Bridge Geotechnical Engineering Report (PBGER)

The purpose of the PBGER is to provide the structural designers with preliminary information that may be used in the preliminary design of the bridge project. Typically, the preliminary report is issued prior to the DFR so that the information can be used during the field review and in development of preliminary bridge plans. The preliminary report should include, at a minimum, the following items:

a. General Project Information
b. General Geology
c. Soils Encountered
d. Seismic
    + Acceleration Design Response Spectrum (ADRS)
    + Liquefaction Screening Results
e. Bridge Foundations
    + Type and Size
    + Preliminary Point-of-Fixity
    + Construction Difficulties
    + Vibration Monitoring Assessment
    + Corrosion Potential Results

Items a through c above should be used to provide a general description of the project and the soils encountered; in depth details are not required. The preliminary seismic information consists of the Acceleration Design Response Spectrum (ADRS) and the results of liquefaction screening. The ADRS and liquefaction screening are conducted in accordance with the procedures provided in Chapter 12. The ADRS may be either preliminary or final depending on the number of testing locations, depth of testing and type of geotechnical exploration. The design seismic event for bridges is based on the classification of the bridge. Typically the Safety Evaluation Earthquake (SEE) is the primary design seismic event for most bridges in South Carolina. However, certain classifications of bridges require both the SEE and the Functional Evaluation Earthquake (FEE) as the design seismic event (refer to SCDOT Seismic Design Specifications for Highway Bridges, latest edition).

The bridge foundations provided in the preliminary report are limited to type and size. Actual capacity determinations are not made until the completion of all field work. If there are limitations in the foundation type, the limitations should be defined as part of the preliminary report; therefore, aiding the structural designer in the final selection of foundations. The preliminary point-of-fixity should be a first estimate based on buckling or a similar procedure. This point-of-fixity is only meant as a starting point in the structural design, not the final point-of-fixity. Any potential construction difficulties, such as anticipated hard driving, should be reported. In addition, a preliminary evaluation of the potential need for vibration monitoring should be reported. The corrosion potential of soils, groundwater and surface water, as applicable, should also be provided in the PBGER for use by the structural designer. The preliminary report typically has a length of approximately two to three pages, not including the Appendix. The Appendix should include the results of field testing related to the bridge, any laboratory work, and the ADRS curve. In addition, a field testing location plan should be provided. For in-house projects, the locations of field tests are typically presented on the Bridge Plan and Profile sheet. In addition, during the preparation of the PBGER, results of hydrometer and grain-size tests are forwarded to the Hydraulic Design Section for use in scour studies for the bridge project.

21.3.2 Final Bridge Geotechnical Engineering Report (BGER)

The BGER is produced after completion of all field work and receipt of loading information from the project designers. The BGER should contain detailed information about the project. This report will be used by the project designers to develop the final design for the bridge project.

The BGER should include a detailed description of the project to include bridge length, structure type, foundation type and size proposed by the structural designer and loading information. Included with the project information should also be a discussion of the existing site conditions at the time of field work. This description should include items such as surface water, exposed rock, vegetation, etc. This is not meant to be a complete list of items describing the actual site conditions. The report should include any items that, in the opinion of the Geotechnical Engineer may affect design or construction. The field and laboratory testing procedures should also be discussed, but this discussion should be limited to the standard test methods used (see Chapter 5). The location of each testing location should also be indicated (see Table 21-1) as well as the type and number of laboratory tests (see Table 21-2). Testing procedures for non-standard tests should be included in the Appendix of the report.

Table 21-1, Soil Testing Location Table

Test Number	Test Hole Local	Station	Offset	Elevation (msl)	Depth (ft)
STB-1	Road	24+99.94	16.84-R	7.24	30
STB-2	Road/Bridge	26+25.52	21.41-R	8.29	80
STB-3	Road	27+27.36	29.23-R	7.07	30
HA-1	Road	23+50.60	17.36-R	6.88	7
HA-2	Road	24+50.18	24.28-R	4.60	1.5
HA-3	Road	25+45.66	23.52-R	9.68	7
HA-4	Road	26+68.55	25.96-R	6.85	7
SCPT-1	Road/Bridge	25+45.03	14.03-R	9.00	14*
DMT-1	Road/Bridge	26+29.19	12.38-R	8.98	16*
MASW/MAM	Road/Bridge	23+75**	13-R**		210

*Depth of refusal.
**Array centered at this station and offset.

Table 21-2, Laboratory Testing Table

Test Type	Quantity
Atterberg Limits	15
Full Sieve Analysis	15
Moisture Content	15
Organic Loss	3
Laboratory Compaction	1
Consolidation	1
Direct Shear	1
Triaxial	1
Corrosion Series	2

The next section of the BGER should consist of discussions of the area geology as it pertains to the overall project, followed by a specific discussion of the soils located at the project site. The overall geologic discussion should include geologic stratigraphic province (see Chapter 11) and geologic formations that will affect the design of the bridge foundations. This discussion should be followed by a detailed discussion of the soils encountered along the bridge alignment. The subsurface conditions should be described as to the type of soil, thickness of each soil type and soil strength as represented by the field work (see Table 21-3). The soil type should be defined based on the field work. Soils that will behave mechanically similarly should be grouped together as a subsurface unit. The thickness may be based on depth below existing grade or based on elevation. The use of elevations is the preferable way of indicating soil thickness. The soil strength may be represented as blow count or other measured indices from the field testing.

Table 21-3, Soil Stratification Table

Geologic Formation	Elevation of Top of Layer (ft msl)	Depth to Top of Layer (ft)	USCS Soil Type	SPT-N values (bpf)	Average CPT Tip Resistance (tsf)	Average DMT p_1 Pressure (tsf)	Comments
Fill	+20	0	SM	3 to 6	73.9	3.8	Existing embankment
Holocene Sediment	+14	6	SM	0 to 1	22.4	3.2	Original ground surface
Cooper	+10	10	SM/ML	12 to 25	138.5	43.7	

After detailing the project information, site conditions and subsurface conditions, the next section of the BGER is the seismic design section. The seismic design section should detail the effects of the design seismic event (see Chapter 12) on the proposed bridge project. A final ADRS should be developed based on the subsurface conditions encountered. The final ADRS will be developed in accordance with Chapter 12. If the site screens for liquefaction during the PBGER, a detailed liquefaction study is required. The liquefaction study will be conducted in accordance with Chapter 13. The detailed liquefaction study should include the extent, both horizontally and vertically, and determination of the amount of induced potential settlement. The potential settlement is accounted for in the determination of downdrag load on interior bent foundations. End bent foundations have more considerations than just downdrag of the fill materials. The lateral stability of the end bent fills must also be addressed. Typically the lateral and vertical stability of the end bent fills is addressed in the RGER, but the effects of the stability should be addressed in the BGER because the end bent foundations will be affected by any instability in the fill sections.

After discussing the geology and seismicity of the project, the next sections of the BGER should address the foundations required to support the structure. The foundation section should include a discussion of resistance factors, including not only the resistance factors, but also how the factors were selected. Both the factored design load and nominal capacities should be provided (see Tables 21-4, 21-5 and 21-6). The type of foundation (shallow footings, piles or drilled shafts) should be provided next, along with the size of the foundations. The depth required to achieve the nominal capacity should be provided along with the minimum depth required to achieve lateral and axial stability. Also to be included in this section of the report are any displacements associated with Service or Extreme Event I limit state loads. This section of the report should also indicate which loading condition (static or seismic) and which stability condition (axial or lateral) governs the selection of foundation size and bearing depth (see Table 21-7).

Table 21-4, Maximum Footing Reaction

Factored Design Load (includes 3 feet of backfill)	295 kips
Factored Net Bearing	4.6 ksf
Geotechnical Resistance Factor	0.45
Required Net Nominal Bearing Resistance	10.2 ksf

Table 21-5, Pile Bearing

Factored Design Load	56 tons
Geotechnical Resistance Factor	0.40
Nominal Resistance	140 tons
Estimated Scour	20 tons
Liquefaction-induced Downdrag	10 tons
Required Driving Resistance	170 tons

Table 21-6, Drilled Shaft Bearing

Factored Design Load	370 tons
Factored Resistance – Side	370 tons
Factored Resistance – End	0
Geotechnical Resistance Factor – Side	0.50
Geotechnical Resistance Factor – End	0.50
Total Nominal Resistance	740 tons

Table 21-7, Governing Conditions

Loading Type	Loading Direction
Static	Axial (Compression or Tensile)
Seismic	Lateral

Construction considerations should follow the discussion of the foundations. This section should address the installation of the foundation and the confirmation of foundation capacity. For driven piles, this section should include preliminary WAVE equation parameters that were used to confirm drivability of the piles. The range of hammer energies should be provided (see Table 21-8). For drilled shafts, any special construction considerations, such as Crosshole Sonic Logging (CSL) tubes, should be included in this section. If shallow foundations are the recommended foundation type to support the structure, this section should include the procedures for verifying bearing capacity.

Table 21-8, Drivability Analysis

Skin Quake (QS)	0.10 in
Toe Quake (QT)	0.08 in
Skin Damping (SD)	0.20 s/ft
Toe Damping (TD)	0.15 s/ft
% Skin Friction	80%
Distribution Shape No.	1
Bearing Graph	Proportional[1]
Toe No. 2 Quake	0.15 in
Toe No. 2 Damping	0.15 s/ft
End Bearing Fraction (Toe No. 2)	0.95
Pile Penetration	80%
Hammer Energy Range	25 – 60 ft-kips

[1]Bearing Graph options – proportional, constant skin friction, constant end bearing

The final section of the report consists of notes that are required to be placed on the plans. The plan notes are project specific but should include foundation capacity tables consisting of factored design loads, nominal resistances, and resistance factors. The required depth to achieve axial or lateral capacity and minimum depth if capacity is achieved prior to required depth shall be provided. This section of the report shall be tailored to the requirements of the specific project and shall provide the information required in Chapter 22. However, in consultant prepared reports, an additional report section is added that consists of any limitations to the report. This section is not required in reports prepared by the GDS.

The Appendix of BGER should include the locations of the soil tests and a subsurface profile. The soil testing reports should be followed by the report of laboratory testing. Only the testing reports that pertain to the bridge should be included in the Appendix. The BGER Appendix should include the final ADRS curve and the results of the detailed liquefaction study, if performed. The results of lateral pile analyses should also be included in the Appendix of the report. For projects performed by the GDS, the lateral pile analysis input screens will be provided and the structural designers will perform the actual lateral pile analysis. For consultant prepared reports, the geotechnical consultant may provide a complete analysis or may provide the input for the analysis depending on the contractual relationship between the geotechnical consultant and the structural designer.

21.4 ROADWAY GEOTECHNICAL ENGINEERING REPORTS

The Roadway Geotechnical Engineering Reports provide geotechnical information related specifically to the design of roadway embankments and any structures other than bridges. The contents of both preliminary (PRGER) and final (RGER) geotechnical engineering reports are described in the following sections.

21.4.1 Preliminary Roadway Geotechnical Engineering Report (PRGER)

The purpose of the PRGER is to provide the designers with preliminary information that may be used in the preliminary design of the roadway embankment and roadway structures. Typically, the preliminary report is issued prior to the DFR so that the information can be used during the field review and in development of preliminary road plans. The preliminary report should include, at a minimum, the following items:

a. General Project Information
b. General Geology
c. Soils Encountered
d. Seismic
    + Acceleration Design Response Spectrum (ADRS)
    + Liquefaction Screening Results
e. Embankment
    + Preliminary slope stability analysis
    + Preliminary settlement analysis
    + Construction Difficulties
    + Temporary Shoring
f. Roadway Structure Foundations
    + Type and Size
    + Construction Difficulties
    + Corrosion Potential Results

Items a through c above should be used to provide a general description of the project and the soils encountered; in depth details are not required. The preliminary seismic information consists of the Acceleration Design Response Spectrum (ADRS) and the results of liquefaction screening. The ADRS and liquefaction screening are conducted in accordance with the procedures provided in Chapter 12 and Chapter 13. The ADRS may be either preliminary or final depending on the number of testing locations, depth and type of geotechnical exploration. In addition, the ADRS shall be provided for both the Safety Evaluation Earthquake (SEE) and the Functional Evaluation Earthquake (FEE).

Preliminary slope stability analysis is based on preliminary road plans and is performed in accordance with the procedures outlined in Chapter 17. The purpose of this analysis is to determine if the designed embankment slope will achieve the required stability or if a flatter slope or reinforcement is required. The settlement analysis should provide an indication of the amount and time anticipated for settlement to occur, because these factors may have an impact on the construction of the project. Any potential construction difficulties, such as mucking/undercutting, should be identified and preliminarily discussed in the PRGER. The need for temporary shoring should also be identified in the PRGER. The preliminary report typically has a length of approximately two to three pages, not including the Appendix. The Appendix should include the results of field testing related to the road embankments, roadway structures and bridge (if present on the project), and any laboratory work. In addition, a field testing location plan and the ADRS curve should be provided.

21.4.2 Final Roadway Geotechnical Engineering Report (RGER)

The RGER is produced after completion of all field work and receipt of revised roadway plans from the project designers based on the results of the DFR. The RGER should contain detailed information about the project. This report will be used by the project designers to develop the final design for the roadway project.

The RGER should include a detailed description of the project to include the roadway length and any roadway structures that may be required to complete the project. Included with the project information should also be a discussion of the existing site conditions at the time of field work. This description should include items such as surface water, exposed rock, vegetation, etc. This is not meant to be a complete list of items describing the actual site conditions. The report should include any items that in the opinion of the Geotechnical Engineer may affect design or construction. The field and laboratory testing procedures should also be discussed, but this discussion should be limited to the standard test methods used (see Chapter 5). The location of each test should also be indicated (see Table 21-1) as well as the type and number of laboratory tests (see Table 21-2). Testing procedures for non-standard tests should be included in the Appendix of the report.

The next section of the RGER should consist of discussions of the area geology as it pertains to the overall project, followed by a specific discussion of the soils located at the project site. The overall geologic discussion should include geologic stratigraphic province (see Chapter 11) and geologic formations that will affect the design of the roadway structures. This discussion should be followed by a detailed discussion of the soils encountered along the roadway alignment. The subsurface conditions should be described as to the type of soil, thickness of each soil type, and soil strength as represented by the field work. The soil type should be defined based on the field work (see Table 21-3). Soils that will behave mechanically similarly should be grouped together as a subsurface unit. The thickness may be based on depth below existing grade or based on elevation. The use of elevations is the preferable way of indicating soil thickness. The soil strength may be represented as blow count or other measured indices from the field testing.

After detailing the project information, site conditions and subsurface conditions, the next section of the RGER is the seismic design section. The seismic design section should detail the effects of the design seismic event (see Chapter 12) on the proposed road project. A final ADRS should be developed based on the subsurface conditions encountered. The final ADRS will be developed in accordance with Chapter 12. The ADRS shall be based on both the SEE and FEE seismic events. If the site screens for liquefaction during the PRGER, a detailed liquefaction study is required. The liquefaction study will be conducted in accordance with Chapter 13. The detailed liquefaction study should include the extent, both horizontally and vertically, and determination of the amount of induced potential settlement. In addition, the potential for lateral spreading and horizontal displacements should also be determined. Where roadways connect with fixed structures (end bents of bridges), the differential movement between the fixed structure and the more flexible roadway will need to be discussed and measures presented to prevent damage to the fixed structure.

After discussing the geology and seismicity of the project, the next sections of the RGER should address the embankments (fills and cuts) and other (i.e. culverts, overhead signs, retaining walls, etc.) roadway structures. Other roadway structures here are typically retaining walls, but may also include culverts (pipe, box or floorless), sound barrier walls and other miscellaneous structures. The results of the stability analyses performed in accordance with Chapters 17 and 18 are discussed. The discussion should include both the static (Service limit state) and seismic (Extreme Event I limit state) performance of the embankment or roadway structure. The results of the analysis should be compared to the performance criteria contained in Chapter 10. If the criteria are exceeded, then site remediation will be required (see Chapter 19). In addition, this section of the report should provide the results of settlement analysis (see Chapter 17). The effects of the settlement should be explained and any measures required to decrease the time for settlement to occur should be provided.

Construction considerations should follow the discussion of the slope stability and settlement analyses. The material properties used during slope stability analysis should be provided in this section (see Table 21-9), because the embankment design is based on these properties being achieved during construction. If ground improvements are required to achieve satisfactory performance of the embankments or roadway structures, the improvements should be discussed in this section. This discussion should include the ground improvement method, the expected result of the improvement and the procedures required for verification of the improvement.

Table 21-9, Fill Material Properties Table

Soil Property	**Required Property**
Internal Friction, φ	32°
Cohesion , c	150 psf
Total Unit Weight, γ	120 pcf

The final section of the report consists of notes that are required to be placed on the plans. This section of the report shall be tailored to the requirements of the specific project and shall provide the information required in Chapter 22. However, in consultant prepared reports, an additional report section is added that consists of any limitations to the report. This section is not required in reports prepared by the GDS.

The Appendix of RGER should include the locations of the soil tests and a subsurface profile. The soil testing reports should be followed by the report of laboratory testing. Only the testing reports that pertain to the embankment and roadway structures should be included in the Appendix. The RGER Appendix should include the final ADRS curves and the results of the detailed liquefaction study, if performed. The results of slope stability analyses should also be included in the Appendix of the report.

21.5 SITE-SPECIFIC SEISMIC RESPONSE ANALYSIS REPORT

The purpose of the SSSRAR (also called a Site-Specific) is to provide the results of a site-specific response analysis. The requirements for conducting a site-specific response analysis are contained in Chapter 12. The SSSRAR should include a detailed description of the project to include bridge length, structure type, foundation type and size proposed by the structural designer and loading information. The results of shear wave testing should be discussed, and included in this discussion should be the testing procedure used to obtain the shear wave velocities.

The next section of the SSSRAR should consist of a discussion of the area geology as it pertains to the overall project, followed by a specific discussion of the soils located at the project site. The overall geologic discussion should include geologic stratigraphic province (see Chapter 11) and geologic formations that will affect the design of the bridge foundations. This discussion should be followed by a detailed discussion of the soils encountered along the project alignment. Also included in this section of the report should be discussion of the project seismicity to include the approximate magnitude and distance from the project site to the anticipated seismic source (M and R, respectively). Included in this section of the report is a discussion of the time histories. Most time histories will be developed using SCENARIO_PC (latest edition); however, actual time histories may also be used. A detailed explanation of the selection of an actual time history shall be provided and should include at a minimum, the M and R and geologic condition where the time history was obtained, along with any scaling required to match the Uniform Hazard Spectrum (UHS) at the project site. The use of actual time histories requires the prior approval of the PCS/GDS.

A detailed liquefaction study and analysis shall not be reported in the SSSRAR, but shall be reported in the BGER or the RGER. The exception to this is if the amount of liquefaction is significant (see Chapter 12) a non-linear analysis is required. The report shall clearly indicate whether a linear or non-linear site response analysis was performed. Included in the report shall be a table indicating the geologic profile to the B-C boundary that was used in the modeling (see Table 21-10). In addition, the sensitivity analysis shall be discussed in detail in this section. The discussion should include which soil parameters were varied and how the variation was determined. As an alternate to varying the soil parameters, multiple seismic events (minimum of seven) may also be used. 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. Use of other methods beyond those included here to develop the ARS curve, requires the prior approval of the PCS/GDS. However, if less than seven seismic events are used an envelope of all ADRS curves is required to develop the final ARS curve.

The Site Class and the three-point ADRS curve shall be determined as if the site-specific analysis was not performed. The three-point ADRS curve and the selected site-specific ARS curve shall be superimposed on each other and the final ADRS curve shall be determined in accordance with Chapter 12.

Table 21-10, 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									

<u>Note:</u> 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).

21.6 SUBMISSION REQUIREMENTS

All reports submitted to SCDOT shall bear the stamp of the Professional Engineer in charge as required by South Carolina law. Two complete, bound, color copies of each report shall be submitted along with at least one CD containing an electronic copy (.pdf). The electronic copy shall also be in color and include all Appendices. The CD containing the electronic copy shall be labeled to include the name of the project, the route or road number, the SCDOT project number, and the name of the geotechnical consulting firm.

Chapter 22

PLAN PREPARATION

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

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CHAPTER 22

PLAN PREPARATION

22.1 INTRODUCTION

This Chapter presents the requirements for the Plan Notes that are required to be included in the final geotechnical reports (BGER and RGER, see Chapter 21) and the preparation of plans specific to geotechnical engineering efforts (i.e., ground improvement). The Plan Notes are prepared by the geotechnical engineer-of-record and included in the Plan Note section of the geotechnical report. The Plan Notes provided in the geotechnical report are required to be placed on the final construction plans.

22.2 PLAN NOTES

Plan Notes are required for bridges, road, ground improvements, geotechnical instrumentation, and earth retaining structures. The list included herein is not meant to be comprehensive of the Plan Notes required. If in the opinion of the geotechnical engineer-of-record additional plan notes are required, the additional Plan Notes shall be provided in the geotechnical report. It is incumbent on the geotechnical engineer-of-record to select and modify the notes presented in this Chapter as appropriate for the specific project. Any Plan Notes that modify or replace a Standard Specification should indicate which Standard Specification is being modified or replaced.

22.2.1 <u>Bridge Plan Notes</u>

The Plan Notes required on bridge plans have traditionally been the most complete and comprehensive notes prepared by the geotechnical engineer-of-record. These Plan Notes cover installation of the foundation (typically, driven piles, drilled shafts or drilled piles). The use of shallow foundations to support bridges is not typical. Plan Notes shall be developed on a project specific basis. The PCS/GDS shall be consulted in the development of shallow foundation Plan Notes.

22.2.1.1 Driven Piles

The following Plan Notes apply to driven piles. Provided first are notes that are general to all driven pile foundations, with the subsequent Sections covering notes specific to particular types of driven piles. It should be noted that Plan Notes are required for both end and interior bents and the general notes should be used accordingly. The notes and tables included herein are generic in nature and should be made project specific. Underlined capital letters are used to indicate areas where project specific information is required. In addition, when the tables presented herein include numbers, these numbers shall be changed to the requirements of specific projects.

Index piles, when used, should be longer than expected production pile length to get full data plus an additional 2 feet if PDA is to be performed. For production piles with PDA, the additional 2 feet required shall be included in the Plan quantities. For piles that are to be driven to grade with no driving criteria due to capacity being achieved by pile “freezing”, some initial driving criteria should be given to insure much softer soils are not encountered and to ensure minimum hammer blow count is achieved.

22.2.1.1.1 General

Place the following notes on the plans for end (interior) bents X and Y:

Table 22-1, Pile Bearing

Factored Design Load	56 tons
Geotechnical Resistance Factor	0.40
Nominal Resistance	140 tons
Estimated Scour	20 tons
Liquefaction-induced Downdrag	10 tons
Required Driving Resistance	170 tons

Method of controlling installation of piles and verifying their capacity: Pile Installation Chart from wave equation analysis without stress measurements during driving.

Method of controlling installation of piles and verifying their capacity: Capacity will be verified by pile driving analyzer and CAPWAP analysis of index pile(s). A Pile Installation Chart developed from the analysis will be used to verify the capacity of production piles.

The required minimum tip elevation to achieve lateral stability for the (PILE TYPE HERE) at end (interior) bent X is M feet MSL.

Perform Pile Driving Analyzer (PDA) testing on the first production pile driven at the end (interior) bent X. Include an additional two feet of (PROJECT SPECIFIC PILE TYPE HERE) length in order to accommodate the initial PDA testing. If a CAPWAP analysis determines that capacity has not been achieved, restrike one of the production piles. Perform the restrike on the production pile exhibiting the least blows per foot. On initial drive, piles shall be stopped at the highest allowable finished grade on the plans to accommodate a restrike while still remaining within an allowable plan finished grade elevation. Perform PDA testing during the restrike. Contact the Bridge Construction Office to determine the time between initial driving and restrike. Payment for the restrike will be as indicated in the Standard Specifications.

Perform Pile Driving Analyzer (PDA) testing on the index pile(s) driven at end (interior) bent X. Drive index pile to grade or to practical refusal, whichever occurs first. If a CAPWAP analysis determines that capacity has not been achieved, restrike the index pile(s). Perform PDA testing during the restrike. Contact the Bridge Construction Office to determine the time between initial driving and restrike. Payment for the restrike will be as indicated in the Standard Specifications.

Each pile is to be installed in one continuous operation. Include details of any anticipated temporary driving discontinuances including anticipated time intervals in the Pile Installation Plan.

Reference the *Standard Specifications for Highway Construction* for Driven Pile Foundations, Section 711. Notes included in these plans are in addition to the requirements of the Standard Specifications.

22.2.1.1.2 Uniform Section Piles

The following notes apply to piles that have a uniform cross section for the entire length of the pile. Included in this group are prestressed, precast concrete piles with a steel H-pile (typically) pile point extending 2-1/2 feet out of the concrete.

The following estimated parameters were used for performing a drivability analysis for end (interior) bents X & Y:

Table 22-2, Drivability Analysis

Skin Quake (QS)	0.10 in
Toe Quake (QT)	0.08 in
Skin Damping (SD)	0.20 s/ft
Toe Damping (TD)	0.15 s/ft
% Skin Friction	80%
Distribution Shape No.[1]	1
Pile Installation Chart	Proportional[2]
Pile Penetration	80%
Hammer Energy Range	25 – 60 ft-kips

[1]Distribution Shape No. varies with depth: 0 at the ground surface (creek bottom); 1 at a depth of 5ft; and 1 to a depth beyond driving depth below the ground surface.
[2]Pile Installation Chart options – proportional, constant skin friction, constant end bearing
Note: GRLWEAP (XXXX) was used to perform the wave equation analysis.

A pile hammer having the rated energy as indicated above is considered suitable for driven pile installation. However, final hammer approval is based on a wave equation analysis that accurately reflects the Contractor's proposed driving system.

The required minimum tip elevation to accommodate critical depth for the (PROJECT SPECIFIC PILE TYPE HERE) is M feet MSL.

22.2.1.1.3 Non-Uniform Section Piles

The following notes apply to combination (composite, non-uniform) piles (i.e., piles that consist of a prestressed, precast concrete pile with a steel H-pile (typically) pile point extending greater than 2-1/2 feet out of the concrete).

The following estimated parameters were used for performing a drivability analysis for end (interior) bents X & Y:

Table 22-3, Drivability Analysis

Skin Quake (QS)	0.10 in
Toe Quake (QT)	0.08 in
Skin Damping (SD)	0.20 s/ft
Toe Damping (TD)	0.15 s/ft
% Skin Friction	80%
Distribution Shape No.[1]	1
Pile Installation Chart	Proportional[2]
Toe No. 2 Quake	0.15 in
Toe No. 2 Damping	0.15 s/ft
End Bearing Fraction (Toe No. 2)	0.95
Pile Penetration	80%
Hammer Energy Range	25 – 60 ft-kips

[1]Distribution Shape No. varies with depth: 0 at the ground surface (creek bottom); 1 at a depth of 5ft; and 1 to a depth beyond driving depth below the ground surface.
[2]Pile Installation Chart options – proportional, constant skin friction, constant end bearing
Note: GRLWEAP (XXXX) was used to perform the wave equation analysis.

The required minimum tip elevation (to accommodate critical depth) for the X-inch prestressed concrete piles is M feet MSL. This equates to a minimum tip elevation of the steel pile points of M feet MSL. Fabricate and deliver piles to the job site with either a 2.5-foot length of HP Y pile point or the full length of HP Y pile point extending from the tip of the prestressed concrete pile.

A pile hammer having the rated energy as indicated above is considered suitable for driven pile installation. However, final hammer acceptance is based on a wave equation analysis that accurately reflects the Contractor's proposed driving system.

The Contractor may elect to drive a portion of the HP Y steel piling extension prior to attaching the remaining prestressed portion. If the Contractor elects to attach the additional extension to the piling prior to picking up the pile, it is expressly understood that the Department is not responsible for any piles damaged during pick-up. Damaged piles are to be replaced at the Contractor's expense.

22.2.1.1.4 Driven Piles - Rock

The previous notes apply to piles driven into soil materials. The following notes should be used for piles driven into intermediate geomaterials (IGM) (i.e., partially weathered rock (PWR)) or to rock. Only one of these notes should be used, it is incumbent upon the designer to determine if reinforced pile tips with teeth are required.

Reinforced pile tips are required to penetrate partially weathered rock. Install the reinforced pile tips in accordance with the manufacturer's installation recommendations.

Reinforced pile tips with teeth are required to penetrate partially weathered rock. Install the reinforced pile tips with teeth in accordance with the manufacturer's installation recommendations. Include the cost of providing teeth on the reinforced pile tips in the price bid for Reinforced Pile Tips.

22.2.1.2 Drilled Shafts

The following Plan Notes apply to drilled shafts. It should be noted that drilled shafts are typically used at interior bents only, but Plan Notes are also required if drilled shafts are used at end bents. Typically the geotechnical designer determines the bottom elevation of the casing, when required. The top elevation of the casing shall be either set by the designer in dry environments or determined by the contractor with approval by the Bridge Construction Office in wet or fluctuating water environments. Regardless of who decides the top elevation of the casing, the top of the casing should not be greater than 5 feet above the existing ground or water surface at the time of construction. The designer should take into account the potential for water variations changing the elevations of the tops of shafts and include any top of shaft elevation restrictions on the plans. The designer shall also provide for permissible construction joints in casings to facilitate construction as applicable on projects with large water elevation fluctuations. The notes and tables included herein are generic in nature and should be made project specific. Underlined capital letters are used to indicate areas where project specific information is required. In addition, when the tables presented herein include numbers, these numbers shall be changed to the requirements of specific projects.

Add the following notes to the plans for Interior Bents X, Y, and Z:

Table 22-4, Drilled Shaft Bearing

Factored Design Load	370 tons
Factored Resistance – Side	370 tons
Factored Resistance – End	0
Geotechnical Resistance Factor – Side	0.50
Geotechnical Resistance Factor – End	0.50
Total Nominal Resistance	740 tons

Assess the actual ground and/or water level conditions and determine the required top of casing elevation and casing length before ordering. Prior to installing any proposed top of casing or top of drilled shaft elevation different from that shown in the plans, obtain approval in writing from the Bridge Construction Office. Support the top of casing to maintain construction tolerances during construction. It is not allowable to extend oversized temporary casing larger in diameter than the construction casing below the scour elevation that is shown on the bridge plan and profile.

If a dry hole is attempted and the hole becomes a wet hole with no slurry equipment on site, immediately backfill the hole with spoils 3 feet up inside the casing or to the top of hole until slurry and desanding equipment are ready on site.

Include the details for anticipated or contingency temporary cessation of work in the Drilled Foundation Installation Plan. Such details shall include shaft maintenance during the temporary cessation.

The estimated bottom of casing elevation and the minimum tip elevation of the drilled shaft are indicated in the table below. The minimum diameter of the drilled shafts is A inches.

Table 22-5, Casing Elevations

Interior Bent No.	Estimated Bottom of Casing Elevation	Minimum Tip Elevation
2	+125 ft msl	+57 ft msl
3	+121 ft msl	+57 ft msl
4	+125 ft msl	+55 ft msl

Reference the *Standard Specifications for Highway Construction* for Drilled Shafts and Drilled Pile Foundations (Section 712) and for Crosshole Sonic Logging of Drilled Shafts (Section 727). Notes included in these plans are in addition to the requirements of the Standard Specifications.

The following notes shall be used if the wet method using mineral slurry is required by the Geotechnical Engineer-of-Record.

Wet construction method for drilled shafts is required. Use mineral slurry throughout the excavation and construction of the shafts. The tolerances for testing (including time intervals) and maintaining the mineral slurries are indicated in the *Standard Specifications for Highway Construction*, Section 712. Do not use plain water, salt water, and/or polymer slurries.

The following notes shall be used as applicable for drilled shafts where a seal is required.

Casing shall be extended into the Cooper Marl (USE APPROPRIATE FORMATION) a sufficient distance to obtain an effective water seal (approximately 1 foot).

Extend casing until the full circumference of the casing penetrates the rock sufficient enough to produce an effective seal against overburden material falling into the shaft. Water may still enter the shaft through seams in the rock.

The following note shall be used for drilled shafts cased to rock where water is an acceptable drilling fluid.

Extend casing until the full circumference of the casing penetrates the rock sufficient enough to produce an effective seal against overburden material falling into the shaft. Water may still enter the shaft through seams in the rock. If the wet method is used, either mineral slurry or potable water may be used during excavation and construction of the shafts. The tolerance for testing (including time intervals) and maintaining the mineral slurry are indicated in the *Standard Specifications for Highway Construction*, Section 712. (INCLUDE ANY REQUIRED WATER TESTS HERE.) Do not use salt water and/or polymer slurries.

The following notes shall be used for drilled shafts placed into intermediate geomaterials (IGM) (i.e., partially weathered rock (PWR)) or to rock.

The estimated bottom of casing and estimated tip elevations for the rock sockets are indicated in the table below. The referenced rock socket penetration depths for Bents X through Y are uncased lengths and the depths indicated are required to be obtained below the top of continuous rock. The minimum diameter for the rock sockets is A inches and the minimum diameter of the drilled shafts is B inches. Support the top of casing to maintain construction tolerances during construction.

Table 22-6, Drilled Shaft Elevations

Int. Bent No.	Estimated Bottom of Casing Elevation	A" Wet & Dry Excavation Per Shaft	B" Rock Excavation Per Shaft	Estimated Tip Elevation
2	+789 ft msl	31 ft	12 ft	+777 ft msl
3	+794 ft msl	25.5 ft	12 ft	+782 ft msl
4	+803 ft msl	15 ft	20 ft	+784 ft msl

During construction, the bottom elevation of the shaft may vary if rock is encountered at a different elevation than shown in the plans. If rock is encountered less than 2 feet higher then the elevation shown, extend the socket to the tip elevation shown. If rock is encountered less than 2 feet lower then the elevations shown, lower the tip elevation as needed to maintain the required depth of rock excavation. If rock is encountered more than 2 feet higher or lower than the elevation shown, immediately notify the Resident Construction Engineer (RCE). The RCE will then immediately notify the Bridge Construction Office.

Provide equipment capable of drilling through rock at the site that may be twenty-five percent (25%) greater than the strength indicated in the Table below.

Table 22-7, Summary of Rock Core Compressive Strength Testing

Boring No.	Recovery	RQD (%)	Core Number	Depth (ft)	Compressive Strength (psi)
B-1	100	41	NQ-2	3 – 4	6,920
B-1	100	68	NQ-4	10 – 10-1/2	5,150
B-3	100	91	NQ-2	31 – 32	6,030
B-4	90	90	NQ-2	57-1/2 - 59	6,840
B-5	100	61	NQ-1	49 - 50	15,620

22.2.1.3 Drilled Piles

The following Plan Notes apply to drilled piles. It should be noted that drilled piles are typically used when the subsurface conditions consist of either IGM or rock at the foundation locations. This type of foundation is used at both interior bents and at end bents. The designer shall take into account that the top of drilled pile concrete will remain 2 feet lower than the temporary casing tip. The notes and tables included herein are generic in nature and should be made project specific. Underlined capital letters are used to indicate areas where project specific information is required. In addition, the tables presented herein include numbers, these numbers shall be changed to the requirements of specific projects.

Table 22-8, Drilled Pile Load Table

Factored Design Load	56 tons
Geotechnical Resistance Factor	0.40
Nominal Resistance	140 tons
Estimated Scour	20 tons
Liquefaction-induced Downdrag	10 tons
Required Driving Resistance	170 tons

The estimated drilled pile tip elevation for End Bent X is M feet below existing grade based on partially weathered rock (CHANGE TO rock AS REQUIRED) at A feet below existing grade (reference soil boring PUT ACTUAL BORING NUMBER HERE). Extend the drilled piles into partially weathered rock a minimum of M feet from top of partially weathered rock. The top of partially weathered rock elevation may be variable across bent location and may result in varying pile lengths. Regardless of the pile lengths, extend the drilled piles M feet into partially weathered rock.

Do not extend the temporary casing for the drilled pile foundations below the top of rock elevation. Prior to concreting and backfilling, remove (pump) any accumulation of water from the excavation. If the hole is a wet hole, concreting using wet construction method may be required as specified in Section 712 (Drilled Shafts and Drilled Pile Foundations) of the *Standard Specifications for Highway Construction*. After installation of the pile, concrete the HP Y piling at End Bent X in the bottom M feet of the rock socket using Class 4000DS concrete. The top of concrete shall remain 2 feet minimum below the temporary casing tip. Wait at least 24 hours after concrete has been placed and then backfill the space between the pile and the excavation with clean sand and tamp in an approved manner. Remove temporary casing used for drilled pile construction. Payment for concrete in the drilled pile foundations is determined using the contract unit bid price for the pay item.

Concrete, backfill and remove temporary casing prior to drilling any adjacent piles within a 20-foot radius.

Reference the *Standard Specifications for Highway Construction* for Driven Pile Foundations (Section 711) and Drilled Shafts and Drilled Pile Foundations (Section 712). Notes included in these plans are in addition to the requirements of the Standard Specifications.

The following notes shall be used if the pile is to be driven after placement in the drilled hole.

The following estimated parameters were used for performing a drivability analysis for End Bents X and Y:

Table 22-9, Drilled Pile Drivability Table

Skin Quake (QS)	0.10 in
Toe Quake (QT)	0.08 in
Skin Damping (SD)	0.20 s/ft
Toe Damping (TD)	0.15 s/ft
% Skin Friction	80%
Distribution Shape No.[1]	1
Pile Installation Chart	Proportional[2]
Pile Penetration	80%
Hammer Energy Range	25 – 60 ft-kips

[1]Distribution Shape No. varies with depth: 0 at the ground surface (creek bottom); 1 at a depth of 5ft; and 1 to a depth beyond driving depth below the ground surface.
[2]Pile Installation Chart options – proportional, constant skin friction, constant end bearing
Note: GRLWEAP (XXXX) was used to perform the wave equation analysis.

Only one of the following notes should be used, it is incumbent upon the geotechnical engineer-of-record to determine which is appropriate.

> Method of controlling installation of piles and verifying their capacity: Pile Installation Chart from wave equation analysis without stress wave measurements during driving. Do not strike the pile over 20 blows with less than 1 inch of pile movement.

> After excavation of drilled pile foundations have occurred, drive the HP Y (change pile type as appropriate) piling to practical refusal. Do not strike the pile over 20 blows with less than 1 inch of pile movement.

Only one of the following notes should be used, it is incumbent upon the designer to determine if reinforced pile tips with teeth are required.

> Reinforced pile tips are required. Install the reinforced pile tips in accordance with the manufacturer's installation recommendations.

> Reinforced pile tips with teeth are required to penetrate partially weathered rock. Install the reinforced pile tips with teeth in accordance with the manufacturer's installation recommendations. Include the cost for providing teeth on the reinforced pile tips in the price bid for Reinforced Pile Tips.

22.2.1.4 Temporary Shoring (Bridge)

The following Plan Notes apply to temporary shoring walls.

> Use buoyant unit weights in computations for soils below the water level. Designer shall determine appropriate water level and consider all unbalanced water forces in design. Design shall accommodate live loading. Use the following soil strength parameters for determining earth pressure coefficients.

Table 22-10, Temporary Shoring Wall Soil Design Parameters

Depth (ft)	c (psf)	Phi (ϕ) (degrees)	Saturated Unit Weight (γ_{sat}) (pcf)	K_o	K_a	K_p
0-7	-	32	100	0.47	0.31	3.25
7-14	343	-	86	1.0	1.0	1.0
14-19	-	30	109	0.50	0.33	3.0
> 19	550	35	120	0.43	0.27	3.69

22.2.2 Road Plan Notes

The Plan Notes required on road plans are a relatively recent development in geotechnical practice as the requirements for design of embankments have increased, leading to a requirement for more detailed notes on the plans.

These notes and tables included herein are generic in nature and should be made project specific. Underlined capital letters are used to indicate areas where project specific information is required. In addition, the tables presented herein include numbers, these numbers shall be changed to the requirements of specific projects. The table provided below presents geotechnical bid items and quantities that should be included in the plans. It is incumbent on the engineer to make sure that all bid items required to assure geotechnical performance have been included.

Table 22-11, Geotechnical Bid Items and Quantities

Item No.	Pay Item	Estimated Quantity	Inclusion Quantity[1]
2034000	Muck Excavation	*200 CY*	*100 CY*
2036000	Geotextile for Separation of Subgrade and Subbase	*362 SY*	*150 SY*
2037000	P7 Geogrid Reinforcement (Uniaxial)	*1944 SY*	*2000 SY*
2037010	P1 Geogrid Reinforcement (Biaxial)	*100 SY*	*150 SY*
2052000	No. 57 Stone as Backfill	*818 TONS*	*180 TONS*

[1]The inclusion quantities associated with mucking and undercutting, i.e. mucking, No. 57 stone, geogrid, and geotextile for separation of sub-grade and sub-base are for bid estimation purposes only. Do not purchase or stockpile these items on site without written approval from the RCE unless specific areas and details are defined on the plans.

22.2.2.1 Borrow Materials

The following notes apply to borrow materials. Underlined capital letters are used to indicate areas where project specific information is required.

Provide borrow materials meeting the following minimum requirements:

- A sandy material with a minimum total soil unit weight, γ_{total} of A pcf
- Minimum friction angle, ϕ, of B° and cohesion, c, of D psf.

In addition, determine the moisture-density relationship and classification of the material. Test and submit the classification, moisture-density relationship, and soil strength parameters of the material to the RPG T GDS for acceptance. An AASHTO certified laboratory is required to perform the testing. Contact the RPG T GDS for a list of locally available AASHTO certified laboratories. The Department may perform independent testing to assure quality.

Determine the friction angle and cohesion using either direct shear testing or consolidated-undrained triaxial shear testing with pore pressure measurements. Classification testing includes grain-size distribution with wash #200 sieve, moisture-plasticity testing and natural moisture content. Use the Standard Proctor test to determine the moisture-density relationship. Remold all samples used in shear strength testing to 95 percent of the Standard Proctor density. Conduct shear strength testing at the initial selection of the borrow pit, any subsequent changes in borrow pits, and for every R cy of materials placed. Perform classification testing for every R cy of materials placed including the material used for the shear strength testing. Additional shear testing may be required if, in the opinion of the RCE, the materials being placed are different from those originally tested.

If these minimum criteria cannot be met, provide the soil parameters for the intended borrow excavation material for the project site to the RPG T GDS for review and acceptance. After acceptable borrow material is obtained, compact the fill to the required finish grade line using the compactive effort indicated in the *Standard Specifications for Highway Construction*, Section 205 (Embankment Construction).

22.2.2.2 Muck Excavation

The following notes apply to muck excavation.

Any areas identified on the plans and any additional areas that are discovered to deflect or settle may require corrective action as directed by the RCE. This may include undercutting, placing No. 57 stone aggregate that is separated from other borrow materials by a geotextile for separation of sub-grade and sub-base, and/or additional compactive effort to the approval of the RCE.

In areas that require mucking or undercutting, borrow material soil may be placed as a bridge lift as long as the grade on which the material is being placed is at least 2 feet above ground water level. In the event that groundwater does not allow backfilling with a borrow material soil, use a No. 57 stone as the bridge lift material. Borrow material bridge lifts may not exceed a 2-foot thickness. The depth at which mucking or undercutting is required is dependent upon encountering a suitable bearing material within the excavation or if a predetermined elevation or depth is required. In most cases, do not undercut more than 3 to 5 feet. The RCE will determine the final mucking or undercutting thickness, unless otherwise specified in the project plans and/or specifications. If a suitable bearing soil is not encountered within this depth range, place a P1 biaxial geogrid with an aperture size of less than or equal to 1 inch beneath a 2-foot thick bridge lift of No. 57 stone. If additional compacted borrow material soil is needed to reach grade, place a geotextile for separation of sub-grade and sub-base between the No. 57 stone and the overlying compacted soil. A bridge lift consisting of borrow material soil may not be placed within 3 feet of the base of the pavement section. Place only compacted borrow material soil or No. 57 stone within this zone. Reference the *Standard Specifications for Highway Construction*, Earthwork Section, Division 200.

Provide biaxial geogrid in accordance with the Special Provisions included in the contract documents.

The quantities associated with mucking and undercutting, i.e. mucking, No. 57 stone, geogrid, and geotextile for separation of sub-grade and sub-base, are for bid estimation purposes only. Do not purchase or stockpile these bid items on site without prior written approval from the RCE unless specific areas and details are defined in the plans.

It should be noted that these notes may also be prepared as a Special Provision, but inclusion on the plans is desirable.

22.2.2.3 Temporary Shoring (Road)

The following Plan Notes apply to temporary shoring walls.

Use buoyant unit weights in computations for soils below the water level. Designer shall determine appropriate water level and consider all unbalanced water forces in design. Design shall accommodate live loading. Use the following soil strength parameters for determining earth pressure coefficients.

Table 22-12, Temporary Shoring Wall Soil Design Parameters

Depth (ft)	c (psf)	Phi (ϕ) (degrees)	Saturated Unit Weight (γ_{sat}) (pcf)	K_o	K_a	K_p
0-7	-	32	100	0.47	0.31	3.25
7-14	343	-	86	1.0	1.0	1.0
14-19	-	30	109	0.50	0.33	3.0
> 19	550	35	120	0.43	0.27	3.69

22.2.3 Ground Improvement

Because there are many different types of ground improvement methods and the notes required for each method are so varied, Appendix E contains a list of template drawings that can be used for the various ground improvement methods. The notes specific to each ground improvement method are contained on the drawings. Copies of template drawings may be obtained from the SCDOT website.

22.2.4 Earth Retaining Structures

Similar to the ground improvement notes, the notes concerning Earth Retaining Structures (ERSs) are placed on drawings depicting the ERS details. ERS notes specific to the particular structure being designed shall be developed on a case-by-case basis.

22.3 PLANS

From time to time, the geotechnical engineer-of-record will be required to or find it necessary to develop plans for use on a specific project (see Appendix E). Typically, the plans developed by the geotechnical engineer are part of the road plan set. Therefore, the plan development procedures presented in the *Highway Design Manual* shall be used in the development of plans. SCDOT only uses 22 inch by 36 inch sheets for plan development. The left margin shall be 2 inches to allow for binding. The remaining margins shall be 1/2-inch. All plans prepared at the direction of the geotechnical engineer-of-record shall be numbered "G-#" for ground improvement and other geotechnical considerations. All plans prepared for ERSs shall be numbered "S-#." The layout of the border shall conform to the latest standard of SCDOT.

22.4 REFERENCES

South Carolina Department of Transportation, *Highway Design Manual*, 2003.

South Carolina Department of Transportation, *Standard Specifications for Highway Construction,* 2007.

Chapter 23

SPECIFICATIONS AND SPECIAL PROVISIONS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

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CHAPTER 23

SPECIFICATIONS AND SPECIAL PROVISIONS

23.1 INTRODUCTION

SCDOT has developed the *Standard Specifications for Highway Construction* to provide requirements, provisions, and direction for use in construction projects. In addition, SCDOT has developed Supplemental Technical Specifications and Supplemental Specifications that modify or supplement the *Standard Specifications for Highway Construction*. The need occasionally arises where modification to the SCDOT specifications or additional specifications are required. The purpose of this Chapter is to provide guidance in preparing these modifications and additional specifications which are Special Provisions for the project.

23.2 SPECIAL PROVISIONS

Special provisions are additions or revisions to the SCDOT *Standard Specifications for Highway Construction*, the SCDOT Supplemental Technical Specifications, and the SCDOT Supplemental Specifications setting forth conditions and requirements for a special situation on a specific project. Special Provisions are included in the contract documents for that project and are not intended for general use. Special Provisions supersede all other contract documents. The designer prepares the Special Provisions for inclusion in the project documents. See Chapter 9 – Bid Documents of the SCDOT *Bridge Design Manual* for additional discussion on the guidelines for preparing Special Provisions.

23.3 REFERENCES

South Carolina Department of Transportation, *Bridge Design Manual*, 2006.

South Carolina Department of Transportation, *Standard Specifications for Highway Construction*, 2007.

Chapter 24

CONSTRUCTION QA/QC

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

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CHAPTER 24

CONSTRUCTION QA/QC

24.1 INTRODUCTION

The purpose of this Chapter is to provide a basic understanding of the construction quality assurance/quality control (QA/QC) as it is applied to geotechnical construction issues. Typically geotechnical construction issues are the verification of foundation capacity, review and acceptance of installation plans, or the review and acceptance of shop plans. Quality Control (QC) is a system of routine technical activities implemented by the Contractor to measure and control the quality of the construction materials being used on a project. Quality Assurance (QA) is a system of review and auditing procedures and testing of a select number of samples by the Department to provide an independent verification of the Contractor's QC program and to provide verification that the construction materials meet the project specifications. QC is performed by the Contractor, while QA is performed by the Department. Ultimately the Contractor is responsible for all materials brought on to a project site; however, it is incumbent on the Department to assure that materials meet Departmental criteria. Construction QA/QC is performed on foundations and some ground improvement installations.

24.2 SHALLOW FOUNDATIONS

Shallow foundations are typically not used to support bridge and roadway structures; however, if shallow foundations are used, contact either the RPG GDS or the PCS/GDS for guidance in developing QA/QC procedures for shallow foundation verification.

24.3 DEEP FOUNDATIONS

24.3.1 <u>Driven Piles</u>

The *Standard Specifications for Highway Construction* requires the Contractor to submit a *Pile Installation Plan* (PIP) for review and acceptance prior to commencing pile installation. The PIP will be submitted to the Department in accordance with the contract documents. On consultant designed projects, the PIP should be forwarded to the design consultant for review. The consultant shall review the PIP for adequacy and for containing the information required by the specifications and plans. The review is to include hammer analysis as described below and should include comments on items such as adjusting hammer fuel settings if needed to protect the pile during driving. Once reviewed, the consultant will return the PIP to the Department with a cover letter containing appropriate comments concerning the PIP. The PIP will be accepted or rejected by the Department. As required, rejected PIPs shall be resubmitted. One of the components of the PIP is the "Pile and Driving Equipment Data Form." Using the information contained on this form, the geotechnical engineer shall perform a Wave Equation Analysis of Pile Driving (WEAP). The WEAP analysis is used to verify that the pile driving hammer should be capable of installing the piles to the correct tip elevation and capacity without inducing excessive stresses in the pile. Piles are typically installed using one of two criteria, capacity or elevation (depth). In some cases, both criteria may be required. Capacity driven criteria is typically based on a required blow count being achieved. The wave equation analysis uses a range of capacities, bracketing the required (nominal) capacity, and range of different strokes.

A typical Pile Installation Chart (also known as a Bearing Resistance Chart or Graph) providing driving criteria is depicted in Figure 24-1.

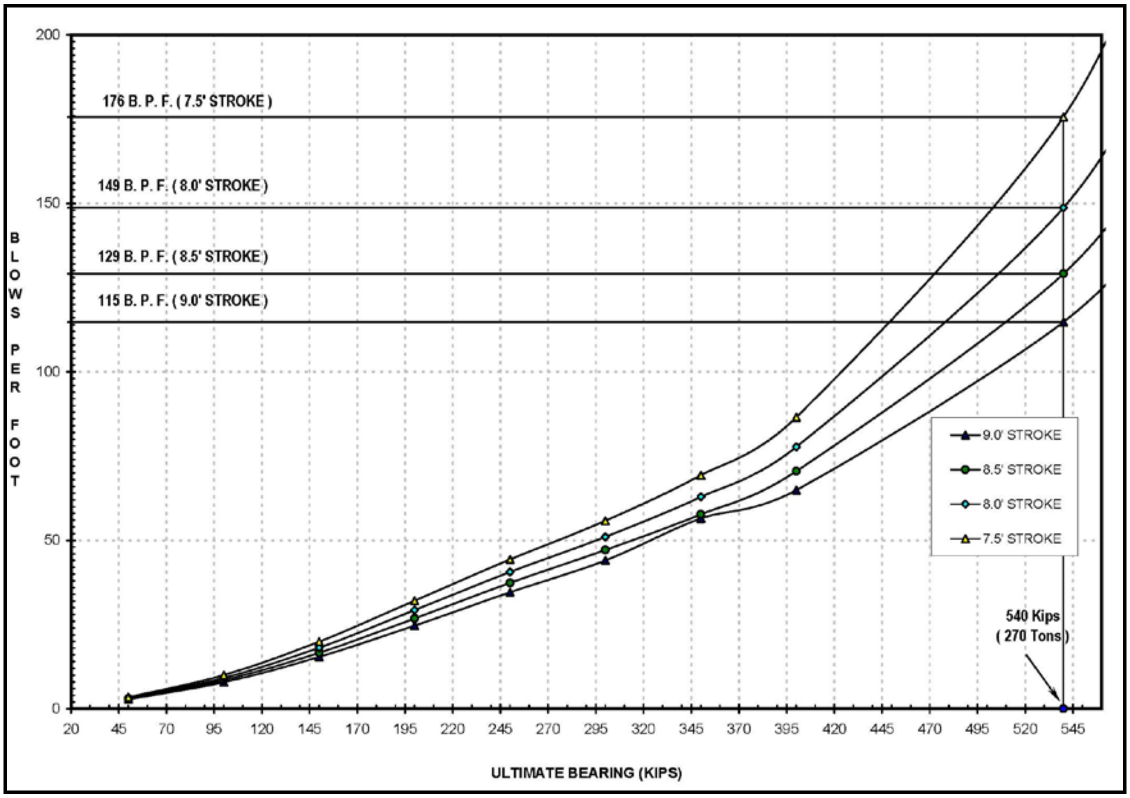

Figure 24-1, Pile Installation Chart

On projects in which the consultant reviewed the PIP, the design consultant shall be responsible for developing Pile Installation Charts as needed as described above or if a PDA or load test is performed as described below. The consultant shall be responsible for recommending pile lengths as needed based on index pile and/or previous production pile data.

The other criterion to control the installation of a pile is elevation (depth). This criterion is used when it is anticipated that the piles will gain strength with time or the lateral stability of the pile controls the tip elevation. Using both criteria, the geotechnical engineer should ensure that the hammer will achieve blowcount criteria as set forth in the *Standard Specifications*. The stresses induced by the hammer should be checked to ensure conformance with the *Standard Specifications*.

Depending on the resistance factor (φ) selected during design, (refer to Chapter 9), load testing may be required. The load testing may consist of high-strain (Pile Driving Analyzer (PDA)), rapid (Statnamic®) or static load. PDA testing may be conducted during initial driving or on restrike or sometimes both. PDA testing can confirm capacity and driving stresses. However, the capacity obtained from PDA testing is approximate and may require further refinement by using Case Pile Wave Analysis Program (CAPWAP). Production piles that are to be tested using PDA should be approximately 2 feet longer than production piles to allow for the attachment of the PDA gauges. Index piles shall be detailed more than 2 feet longer than production piles to get full driving data to be used in determining pile lengths. Using the results from CAPWAP, a second WEAP analysis should be performed to more accurately model the installation of the pile. This is especially important when the pile is being driven to a specific capacity at a specific blow count. Monitoring of stresses using the PDA is critical when piles are installed into or through dense formations or partially weathered rock (PWR), or through very soft formations.

Statnamic® and static load testing are performed after the installation of the pile. These tests are normally performed prior to production pile driving. Statnamic® is a rapid load test and is different from the PDA, in that the pile is subjected to a “fast push” rather than a sharp blow as would be observed from a pile hammer. Statnamic® load testing of piles can be relatively costly, especially given the capacity requirements. Static load testing, if required, follows the standard testing method developed and presented in ASTM. Static load testing can not only be expensive, but also time consuming, and is therefore not used except in design testing programs. When static load testing is performed, the results of the testing shall use the Davisson failure criterion.

24.3.2 Drilled Shafts

Similarly to driven piles, the *Standard Specifications for Highway Construction* requires the Contractor to submit a *Drilled Foundation Installation Plan* (DFIP) for review and acceptance prior to commencing drilled foundation installation. The DFIP will be submitted to the Department in accordance with the contract documents. On consultant designed projects, the DFIP should be forwarded to the design consultant for review. The consultant shall review the DFIP for adequacy and for containing the information required by the specifications and plans. Once reviewed, the consultant will return the DFIP to the Department with a cover letter containing appropriate comments concerning the DFIP. The DFIP will be accepted or rejected by the Department. As required, rejected DFIPs shall be resubmitted. To verify the acceptability of drilled shafts, crosshole sonic logging (CSL) tubes shall be installed as required by the *Standard Specifications for Highway Construction*, project plans or Special Provisions. If the CSL testing indicates no areas of concern, then the drilled shaft is accepted. However, if the CSL testing indicates areas of concern, then the following forms should be requested by the geotechnical engineer for review:

- Drilled Shaft Log
- Drilled Shaft Excavation Log
- Slurry Inspection Log
- Drilled Shaft Inspection Log
- Drilled Shaft Concrete Placement Log
- Drilled Shaft Concrete Volumes Log
- Concrete Slump Loss Test

After reviewing these logs, the geotechnical engineer should consult with the Bridge Construction Engineer on required actions. On projects in which the consultant reviewed the CSL results, the design consultant shall be responsible for evaluating the drilled foundation to determine if it meets both the structural and load resistance design requirements. The consultant shall make written recommendations to the Department concerning drilled foundation acceptance and/or actions to be taken.

Depending on the resistance factor (φ) selected during design (refer to Chapter 9), a load test may be required. Load tests can be used to verify the existing design or modify the design based on the load test results. On consultant designed projects, the consultant should evaluate the test results and provide written recommendations to the Department concerning the diameter, penetration depth relative to a particular stratum, and/or tip elevation of the production shafts. Drilled foundation load testing consists of static (uni-directional and bi-directional), rapid (Statnamic®) or high-strain (dynamic) load testing. The resistance factor for high-strain load testing shall be the same as for rapid (Statnamic®). In a uni-directional static load test, the load is applied at the top of the drilled shafts, usually by means of a reaction beam and anchorage foundations. Typically, this type of test is impractical for drilled shafts, unless the drilled shafts have diameters ranging from 3 to 4 feet. Typically drilled shafts in this range have nominal capacities of 600 tons, which require a reaction system and jack to have 1200 tons of capacity. Drilled shafts having larger diameters would require very large reaction systems that would become impractical and potentially unsafe. Therefore, uni-directional static load tests are not normally performed on drilled shafts. Bi-directional static load tests are performed by applying the load with an expendable jack(s) located between an upper and lower loading plate cast into the drilled shaft. The test is conducted by using the upper portion of the shaft as a reaction element against the base and lower portion of the drilled shaft and vice versa. An effective example of this bi-directional loading system is the Osterberg cell (O-cell) (see Figure 24-2). The maximum test load is limited by the resistance of the shaft above and below the O-cell or the maximum capacity of the O-cell. Therefore, the O-cell should be placed at the point in the shaft where the capacity above the O-cell is approximately equal to the capacity below the O-cell. The use of multiple O-cells may be used to counter the effect of either too much side resistance or too much end resistance, when compared to end or side resistance respectively (see Figure 24-3). The Davisson failure criterion shall be used to interpret the results of uni-directional and bi-directional static load tests.

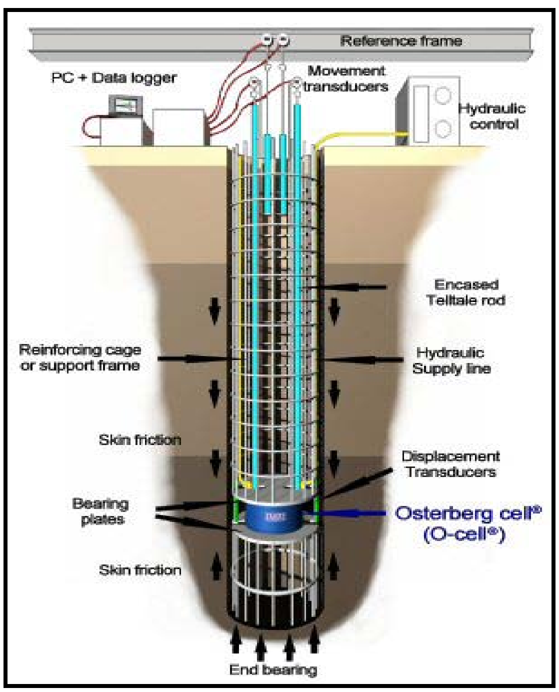

Figure 24-2, Bi-Directional Testing Schematic (LOADTEST, Inc.)

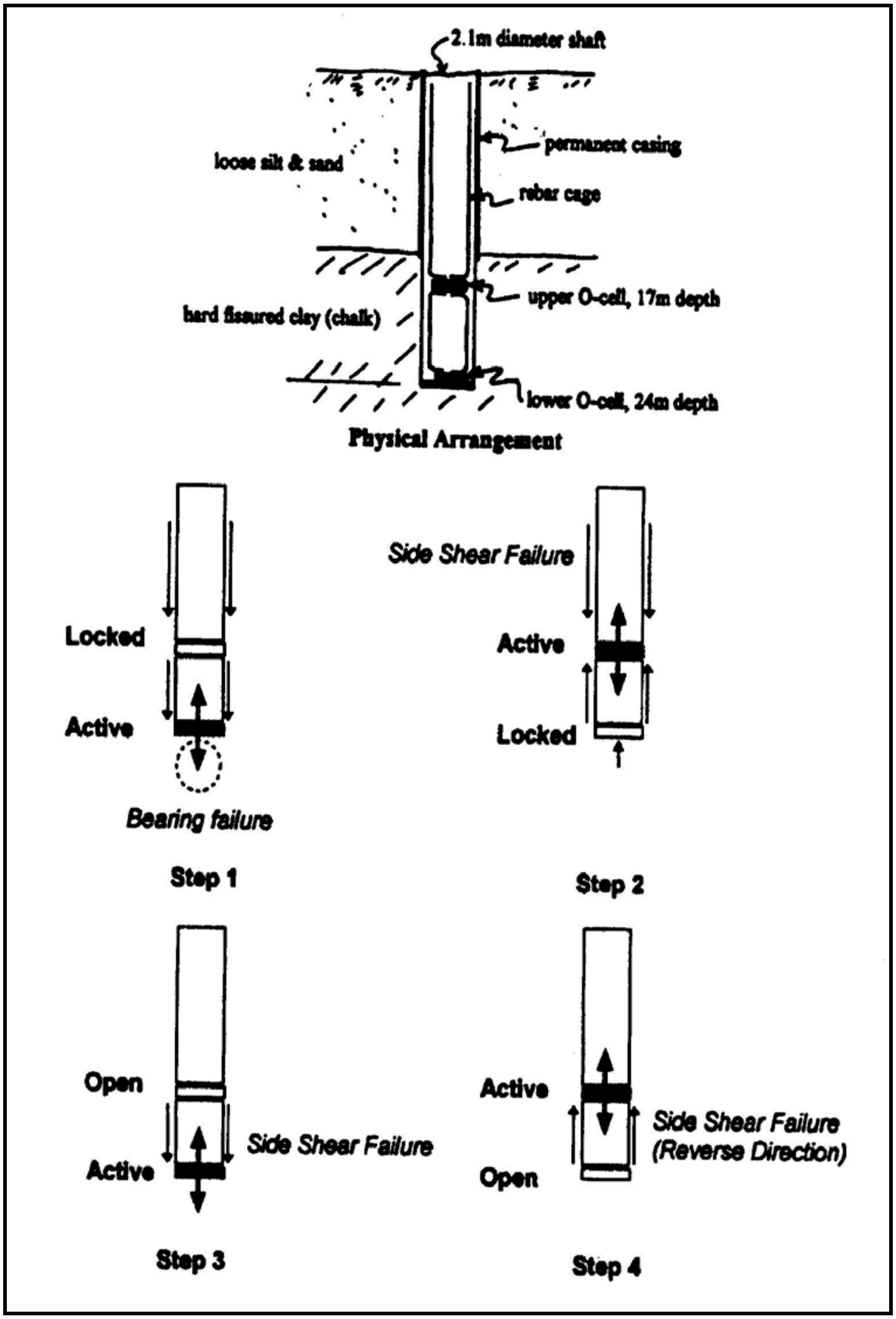

Figure 24-3, Multiple O-cell Arrangement
(O'Neil and Reese, 1999)

Listed below are some advantages of the bi-directional static load test:

- Large capacity allows testing of production-sized shafts
- With multiple cells or proper instrumentation, the base and side resistance are isolated from the resistance of other geomaterial layers
- Loading is static and can be maintained to observe creep behavior

Following are some of the disadvantages of bi-directional static load testing:

- The test shaft must be preselected so that the O-cells can be included
- It is not possible to test an existing shaft
- For each installed device, testing is limited to failure of one part of the shaft only, unless multiple O-cells are used
- The performance of a production shaft subject to top down loading must be computed and may require extrapolation of data in some cases
- Limitations exist related to using a test shaft as a production shaft
- The effect of upward directed loading compared to top down loading in a rock socket is not completely understood

Rapid load testing is between static load testing and high strain testing of drilled shafts. In rapid load testing the drilled shaft is subjected to a "fast push" instead of a sharp blow as would be delivered by a pile driving hammer or a falling weight. In a rapid test, the drilled shaft acts essentially as a rigid body with the top and base of the shaft moving together.

There are two methods of inducing load during the rapid test. The first method consists of dropping a weight onto the shaft, but having a soft cushion located at the top of the shaft. The soft cushion causes the weight to decelerate over a required time interval. The second method, and most common, is to accelerate a heavy mass upward using combustion gas pressure, thus pushing the shaft into the ground. Using the second method, commercially available as the Statnamic® load test apparatus, a reaction mass is accelerated vertically while an equal and opposite reaction occurs in the drilled shaft.

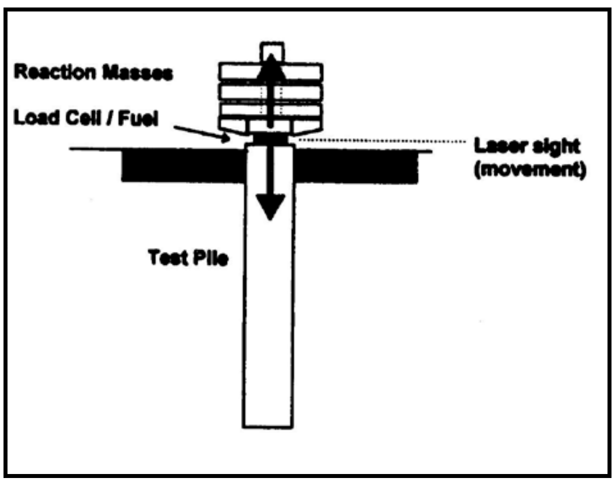

Figure 24-4, Rapid Load Test Setup (O'Neil and Reese, 1999)

Listed below are advantages of the rapid load test method:

- Test capacities up to 5000 tons (Statnamic®)
- Can test existing or production shafts
- Economies of scale for multiple tests
- Easily used for verification testing on shafts
- Reaction system not needed

Some disadvantages are:

- High capacity, but still limited compared to bi-directional tests
- Rapid loading method, rate effects must be considered
- Mobilization costs for reaction weights

High-strain load testing of drilled shafts uses the same equipment and principles as PDA testing and CAPWAP analysis in driven piles. High-strain load testing uses a hammer or weight to strike the top of a shaft inducing a compression wave that propagates the length of the shaft and reflects back to the top. The impact load can be induced using drop weights (see Figure 24-5) or a large pile driving hammer. If suitable measurements are obtained, then the applied load and pile response can be determined. The measurements are obtained using transducers and accelerometers mounted directly to the top of the shaft. A computer model of the shaft response to the blow is calibrated to the measurements using a signal matching technique (i.e., CAPWAP). The high-strain dynamic load test setup should always be modeled prior to testing using a wave equation model for specific shaft size and axial capacity. Because the high impact velocity can produce significant compression and tension forces in the shaft, the blow is typically cushioned using a cushioning material such as plywood or a striker plate.

Figure 24-5, High-Strain Load Testing Apparatus (GRL Engineers, Inc.)

Listed below are advantages of high-strain load testing:

- Large load capacity, applied at top of shaft
- Can test existing or production shafts
- Economies of scale for multiple tests
- Easily used for verification testing on production shafts
- Reaction system is not needed

Some disadvantages are:

- Capacity high, but still limited compared to bi-directional tests
- Test includes dynamic effects which must be considered
- The applied force is interpreted from measurements on the shaft rather than from direct measurement of load and therefore is sensitive to the shaft modulus, area, and uniformity in the top 1 to 1-1/2 diameters
- Test must be designed to avoid potential damage to the shaft from driving stresses
- Mobilization costs for a large pile driving hammer or drop hammer
- Changes in impedance along the length of the shaft can be confused with changes in axial resistance, and therefore the impedance profile of the shaft must be reliably known.
- There may be incomplete mobilization of base resistance at early blows and loss of side resistance after multiple blows, and this issue complicates the interpretation of results.

24.4 GEOTECHNICAL INSTRUMENTATION

This Section discusses the interpretation of the results of geotechnical instrumentation; Chapter 25 discusses the requirements of the instrumentation, location of the instrumentation and monitoring of the instrumentation. The results obtained from geotechnical instrumentation require QA/QC to in order to determine if the data is meaningful. On projects in which the consultant reviews the results of geotechnical instrumentation, the design consultant shall be responsible for evaluating the geotechnical instrumentation data to determine if it meets the design requirements. The first questions concerning the results of geotechnical instrumentation is: "Was the data collected in a manner consistent with the plans, specifications and special provisions?" If the data was not collected in a consistent manner, every effort should be made to determine why not. If the data collected is consistent, next check to determine the numerical accuracy. Finally, the data should be checked for consistency with previous data. If the data is not consistent, does a hypothesis exist that explains all the data? If not, then consideration should be given to the point that the data is bad and should be discarded. The following Subsections will briefly discuss the interpretation of data collected from the various forms of geotechnical instrumentation.

24.4.1 Slope Inclinometers

The review of inclinometer data should indicate first that the bottom of the casing is placed firmly in material that is not moving (i.e., below the potential/actual failure surface). Second, the review should indicate that all subsequent data is indicating movement "down hill." If the data indicates movement in the opposite direction, review the procedures for obtaining the data with field personnel. In addition, the actual movement data should be compared to the theoretical (design) movements to determine if the predicted is similar to the actual. From this comparison, it may be possible to predict additional movements.

24.4.2 Settlement Monitoring

The monitoring of settlement is probably the most common type of geotechnical instrumentation used by SCDOT. A variety of methods are used to monitor settlement (see Chapter 25). Typically settlement data consists of either survey (elevation) data or pore pressure data. Survey data is obtained from various points that are compared to established benchmarks, while pore pressure data is obtained from piezometers. This Section discusses the evaluation and reduction of the data, not the collection. The first check of the data is to determine if the numerical calculations are consistent. The second check and more important check, is the trend of the data, i.e. does the data continue to indicate downward movement. With pore pressure data, the second check is whether or not the pore pressures are approaching a static pore pressure level. It should be noted that the before construction pore pressure level will not be obtained, but some higher level will be. Both the survey data and the pore pressure data should approach a trend line where there is very little difference between readings. Once this happens, settlement is assumed to be over. While settlement monitoring is occurring, the amount of actual settlement should be compared to the predicted amount of settlement. One method for determining if settlement (based only on survey data) is complete is to use Taylor's square root of time method.

24.4.3 Tiltmeters

The tiltmeter data should be reviewed to ascertain if any movement has occurred. If movement has occurred, does there appear to be a trend of continuing movement, or has the movement stopped? Tiltmeters should be left in place long enough to determine if the tilt reverses during seasonal changes.

24.4.4 Vibration Monitoring

According to <u>Ground Improvement Methods</u>, "The U.S. Bureau of Mines has found that building damage is related to particle velocity. Figure *24-6* was developed by the Bureau based on experiences with damage measurements made in residential construction from blast-induced vibrations. The limiting particle velocity depends upon the frequency of the wave form. Normally, *construction vibrations may* result in frequencies of 5 to 12 *Hertz* (Hz). Using Figure 24-6 as a guide, this would limit peak particle velocities to values of ½-inch per second for older residences with plaster walls and ¾ inches per second for more modern constructions with drywall. Peak particle velocities that exceed the values given in Figure 24-6 do not mean damage will occur. Rather, these values are the lower threshold beyond which cracking of plaster or drywall may occur.

"Data generated by the U.S. Bureau of Mines indicate that minor damage occurs when the particle velocity exceeds 2 inches per second (51 mm/sec), and major damage occurs when the particle velocity exceeds about 7-1/2 inches per second (190 mm/sec). Thus, keeping the particle velocity less than about ½ to ¾ inches per second should be a reasonably conservative value to minimize damage."

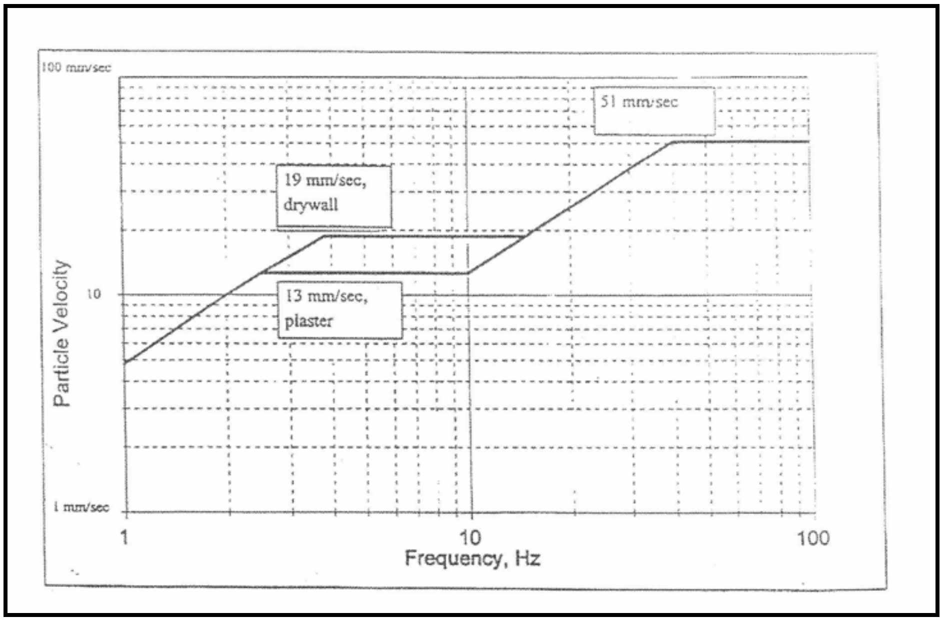

Figure 24-6, Safe Level of Vibrations for Houses (Ground Improvement Methods – September 2005)

Seismographs are typically used to measure ground velocities caused by dynamic operations (deep dynamic compaction, pile driving, etc.). Typically a base line reading is obtained prior to commencing operations to obtain the level of ambient background vibrations. The readings during production operations are obtained from seismographs on adjacent structures or at the construction limits. However, prior to construction operations an estimate of the particle velocity to be generated is required. Figure 24-7 can be used for planning purposes.

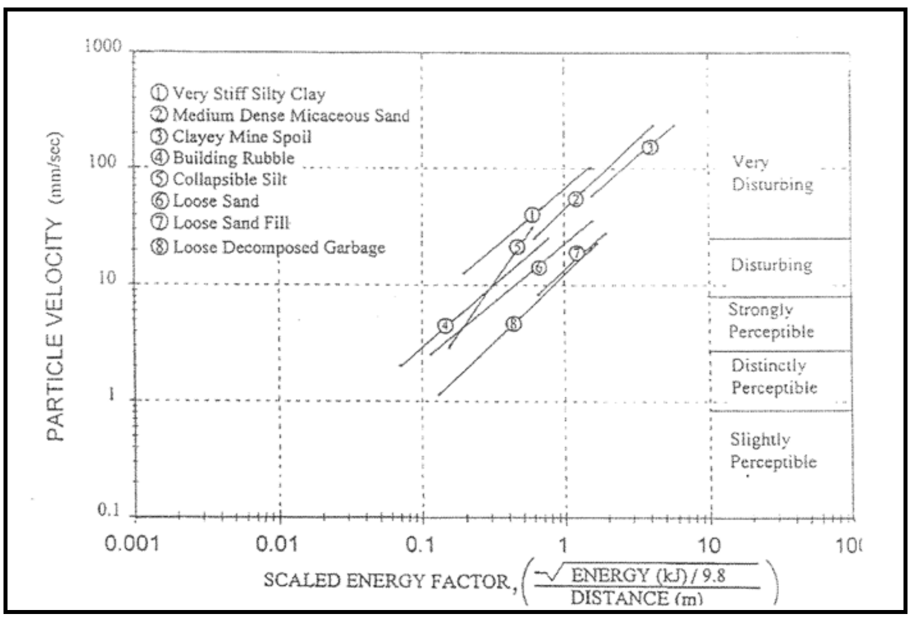

Figure 24-7, Scaled Energy Factor vs Particle Velocity (Ground Improvement Methods – September 2005)

kJ = 737.5 ft-pounds; m = .3048 ft; mm = 0.0394 inches

If the estimated particle velocity exceeds the project requirements, then alternate construction activities may be required. Ground vibrations on the order of ½ to ¾ inches per second are perceptible to humans. Even though these vibrations should not cause damage, vibrations of this magnitude can lead to complaints. The adjacent property owners should be educated concerning the potential impacts of the ground vibrations should be performed.

24.5 SHOP PLAN REVIEW

The Standard Specifications, Special Provisions, Supplemental Specifications and design drawings occasionally require the Contractor to submit Shop Plans and Installation Plans in addition to the PIP and the DFIP. The geotechnical engineer-of-record shall review the geotechnical portions of the submitted Shop Plans and Installation Plans for conformance to the Standard Specifications, Special Provisions, Supplemental Specifications and design drawings. If no review time is specified in the contract, then the geotechnical engineer-of-record shall conduct the review in twenty-one calendar days and shall submit the response to the Department.

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Chapter 25

CONSTRUCTION MONITORING AND INSTRUMENTATION

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

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CHAPTER 25

CONSTRUCTION MONITORING AND INSTRUMENTATION

25.1 INTRODUCTION

This Chapter provides a general overview of the selection and use of geotechnical instrumentation on SCDOT construction projects. There are two general classes of geotechnical instrumentation. The first class is those instruments used to investigate and evaluate soil and rock properties. This class of geotechnical instrumentation is presented in Chapter 5 – Field and Laboratory Testing Procedures. The second class is those geotechnical instruments that monitor performance during and after construction. Excluded from this Chapter is the instrumentation of deep foundations (see Chapter 24), surveying, and construction materials testing. This Chapter is not intended to provide specifications for individual instruments, but rather to provide a systematic approach to the planning for and implementation of an instrumentation and monitoring plan. For more specifics regarding the information presented herein, please refer to the references cited at the end of this Chapter. This Chapter will cover typical types of field instrumentation used on SCDOT highway construction projects.

Field instrumentation on highway projects can play several vital roles, including the following:

- Verification of Design Parameters – Data obtained from instrumentation can be used to verify that the constructed embankment, slope, wall, etc. behaves as predicted during and after construction. Initial data can be used to modify the design if necessary.

- Evaluate Performance During Construction – Field instrumentation can be used to monitor construction performance of the embankment, slope, wall, etc. that may affect or be affected by construction activities and that may affect the construction schedule.

- Evaluate Performance of Existing Structures – Existing embankments, slopes, walls, etc. can be instrumented to assess the existing conditions and to guide remediation measures, if necessary.

- Detect short and long-term trends – Before potential problems are visible to observers, instrumentation can provide the first indication of how a structure is going to perform over short-term and long-term periods.

- Safety – Field instrumentation can serve as the first warning sign of a potentially unsafe situation. An instrumentation and monitoring program can also play a role in easing public concerns over safety of areas surrounding the construction site.

- Legal Protection – Instrumentation can provide documentation as to the relationship between construction and surrounding structures. In the event of litigation, data from these instruments can be used to prove/disprove connection of damage in surrounding areas to construction activity.

Planning an instrumentation and monitoring program should be guided by a systematic approach. The steps listed in this Chapter provide a typical list of planning considerations that can be applied to most highway construction projects. The overall objective for the program should be decided before selection of instruments commences. As part of the planning process, the need for instruments should be gauged against such factors as relevance of the data obtained, impedance of construction, and cost.

Although the goal of this Chapter is not to provide specific guidelines on field instrumentation, the general philosophy given in the references cited below should be applied to nearly every project where field instrumentation is to be used. First, every instrument should be installed to answer a specific question. More instrumentation than is required produces additional, perhaps harmful, discontinuities in the structure and may provide a false sense of security. Second, in general the simpler the instrumentation is, the more desirable it should be. Although some situations may arise where sophisticated instrumentation cannot be avoided, such as the need for remote monitoring, simpler instruments generally provide data that is just as reliable while having less chance of malfunction, and at a reduced cost. Third, redundancy, or a system of checks, should always be built into the monitoring program to add another level of reliability beyond what is provided by a single instrument. If sophisticated instruments are to be used, standard, "low-tech" instruments can also be installed to maintain the flow of incoming data in case of malfunction in the sophisticated instruments.

25.2 MONITORING PLAN

Before field instruments are to be installed, a thorough monitoring plan should be submitted for review by the geotechnical engineer-of-record. The elements to be included in this plan are detailed below and generally follow the guidelines set forth in Geotechnical Instrumentation (FHWA-HI-98-034) dated October 1998. Table 25-1 provides a list of the elements used in developing a monitoring plan.

Table 25-1, Monitoring Plan Elements

1. Definition of Project Conditions	2. Objectives of Instrumentation
3. Predicted Magnitude of Change	4. Define Remedial Actions
5. Establish Responsibilities and Chain of Command	6. Types of Instruments and Locations
7. Recording of Outside Factors	8. Procedures for Ensuring Data Validity
9. Estimate Costs	10. Installation and Protection Plans
11. Calibration and Maintenance of Field Instruments	12. Data Processing

The monitoring plan as well as certain construction related items, such as monitoring, calibration and maintenance, data collection, processing, presentation, interpretation and reporting, are considered "professional services" and should not be left to the Contractor to perform. The monitoring plan shall be prepared by the geotechnical engineer-of record, whether by SCDOT personnel or the geotechincal consultant. On most SCDOT construction projects geotechnical instrumentation is installed by the Contractor under the supervision of a licensed engineer. In addition, in many cases the monitoring, calibration and maintenance and data collection are made the responsibility of the Contractor. In these cases, the Contractor shall be required to retain the services of a licensed engineer, familiar with the instrumentation being used. The processing, presentation, interpretation and reporting are typically provided by the geotechnical engineer-of-record, whether by SCDOT personnel or the geotechnical consultant.

25.2.1 Definition of Project Conditions

This Section of the instrumentation plan should include a summary of existing conditions and of proposed construction, if applicable. If a Geotechnical Base Line Report has been completed for the site to be monitored, a short summary of the relevant information from this report should be included in the monitoring plan. Other information that may be relevant to monitoring, such as condition surveys of existing structures or reports of environmental conditions, should also be summarized in the monitoring plan. All pertinent information about the project related to the monitoring program should be properly referenced in the monitoring plan. If additional information is needed to fully characterize the site, a plan for obtaining this information shall be submitted with the monitoring plan.

25.2.2 Objectives of Instrumentation

The objectives of field instrumentation to be used on the project shall be clearly defined in the monitoring plan. The first step to defining objectives for field instrumentation is to predict potential failure mechanisms that may occur during or after project completion. Secondly, instruments can be installed to monitor parameters such as pore water pressure, horizontal and/or vertical displacements, in-situ stresses, etc. that are indicative of a failure. Finally, the information gained from the field instruments shall be used to support any further action that may be necessary. If the objectives of the instrumentation cannot be clearly defined, delete the instrumentation. Only use instrumentation that have clearly defined objectives.

25.2.3 Predicted Magnitudes of Change

The lower bound of predicted magnitudes will provide the required accuracy of field instruments, while taking into account the full range of predicted magnitudes will convey the required data range of field instruments. Threshold levels which correspond to escalating need for remedial action shall also be determined and included with the monitoring plan. A table or similar graphic illustrating these levels should be displayed in a prominent place and all personnel associated with monitoring shall be aware of both the threshold level readings and required remedial actions. The threshold values are chosen based on experience with similar projects, similar subsurface conditions or construction methods, case histories of similar projects, and engineering judgment of project personnel.

25.2.4 Define Remedial Actions

In relation to threshold levels, remedial actions corresponding to each escalating level shall be defined in the monitoring plan. Remedial actions will be project specific but may range from simply informing someone higher up the chain of command of a possibly unsafe situation, to stopping work, or to emergency measures in the event of an impending failure. A detailed description of each action may not be feasible at the time the plan is written, but the plan shall at least describe each action in general terms. Pre-project planning ensures that the required labor and materials will be available in case of emergency.

25.2.5 Establish Responsibilities and Chain of Command

The responsible parties for each phase of a monitoring program, from planning to collection and interpretation of data, shall be designated either in the monitoring plan or in another suitable document. Responsibilities and authority of each party in relation to the other parties regarding the monitoring program shall also be clearly defined. Regardless of their role and level of authority on the project, monitoring personnel shall always have a direct line of communication between themselves, the construction Contractor, and the geotechnical engineer-of-record in case a situation arises that needs immediate attention.

25.2.6 Type of Instruments and Locations

The type, number, and manufacturer of each instrument to be used on the project shall be provided in the monitoring plan. The reasons for selecting particular instruments to monitor the conditions described above shall also be explicitly spelled out, keeping in mind that every instrument is installed to answer a specific question. The overriding factor in choosing field instrumentation is reliability. Other factors such as ease of installation, difficulty of interpretation, and cost, may also play a role. Instrument manufacturers can provide valuable information during the instrument selection process about relevance of the instrument to the specific application and limitations of the instrument.

The locations for instrument installation shall be chosen based on potential failure analysis, preexisting information (if for an existing structure or slope), subsurface conditions, and any other pertinent information. If site conditions are generally homogenous, instruments may be installed at selected intervals. If it appears that certain areas will be more critical or have a higher probability of failure, instruments shall be concentrated at these locations. Provisions should be made to order more instruments than necessary to account for damage during installation or malfunction once the instrument is installed. Field instrument locations shall be clearly marked on a plan view of the site. Instrumented cross-sections, if applicable, shall also be included with the monitoring plan.

25.2.7 Recording of Outside Factors

The recording of all outside factors, that can be reasonably assessed, that may influence field instrument data shall be specified in the monitoring plan. This is especially important for monitoring during construction activities, as heavy construction traffic and altering of the site conditions can have a significant effect on instrument data. Monitoring personnel must keep or have access to a detailed record of construction activities in order to correlate monitoring results and filter out anomalies caused by nearby construction activities. Other outside factors that may influence instrument readings include environmental conditions such as temperature, rainfall, sunlight, and seismic activity.

25.2.8 Procedures for Ensuring Data Validity

Procedures shall be in place to ensure the validity of each instrument installed for the project. Redundancy is an effective way to reduce error in instrument data. For example, an open-standpipe piezometer can be installed near a pore-pressure transducer, screened at the same interval, to ensure that pore pressure readings are accurate. Optical or GPS surveying of surface monuments can be used to validate apparent movements indicated by subsurface instruments. Visual observation of site conditions by trained personnel can also be an effective means of validating instrument data. Systematic checks of data reliability should be planned for each type of instrument to be installed.

25.2.9 Estimated Costs

An estimated cost tabulation sheet for both materials and labor associated with the proposed monitoring procedures shall be compiled and submitted either with the monitoring plan or with another suitable document. Contingencies shall also be put in place to cover additional monitoring should the need arise.

25.2.10 Installation and Protection Plans

A detailed set of installation plans, including at least a work plan and sketches, shall be included with the monitoring plan. Oftentimes, the instrument manufacturer will provide detailed installation plans for their instruments. If necessary, the appropriate ASTM or AASHTO standard shall be referenced with regard to installation. Included with the installation plan shall also be methods to assure that the instrument is installed correctly and for the initial calibration of the instrument. If the instrument is to be installed in an active construction zone, plans must include methods for handling, protecting and repairing the instrument.

25.2.11 Calibration and Maintenance of Field Instruments

The instrument manufacturer is required to provide a recommended schedule for calibration and maintenance of field instrumentation. A calibration schedule of at least once per year is recommended, although many instrument manufacturers recommend shorter time periods between calibrations. Periodic calibration checks should also be performed by monitoring personnel to ensure that the instruments remain in calibration throughout the life of the project.

25.2.12 Data Processing

The procedures to be used for data collection, processing, presentation, interpretation, reporting, and implementation shall be provided in the monitoring plan. Field instrument reading schedules shall be spelled out in the monitoring plan, but must remain flexible depending on project progress and the results of initial readings. The plan shall also indicate specific software that may be required for processing data. Typically, field instruments are read on a relatively tight schedule at the beginning of a project and then relaxed as baseline conditions emerge and/or the project progresses beyond critical stages. Management of instrument data from methods of field collection to data storage and backup shall be accounted for in the planning stages of the project. The time needed for post-processing of instrument data will be dependent on instrument type and level of sophistication. Sufficient effort shall be planned for data interpretation by trained personnel. The results of data analysis shall be provided in periodic reports corresponding either to a set time interval (i.e. weekly, monthly, etc.) or to project milestones.

25.3 MONITORING PLAN EXECUTION

As discussed previously, the installation of geotechnical instrumentation is the responsibility of the Contractor. The Contractor shall be required to submit an installation plan for review. The plan should include the items in Table 25-2.

Table 25-2, Monitoring Plan Execution

1. Instrumentation Supplier	2. Factory calibration of instrumentation
3. Pre-Installation testing requirements	4. Calibration and Maintenance Requirements
5. Installation methods	6. Protection plan
7. Installation records	8. Installation report
9. Data Collection methods	10. Qualifications of personnel collecting data

25.3.1 Instrumentation Supplier

The Contractor shall be required to provide the name of the supplier of the geotechnical instrumentation and all literature provided by the supplier. The literature shall be used to verify that the instrumentation selected meets the requirements of the project.

25.3.2 Factory Calibration of Instrumentation

All instrumentation shall be calibrated at the factory prior to shipment and calibration certificates shall be provided by the Contractor. Any additional calibration requirements contained in the Special Provisions or Supplemental Specifications shall also be met.

25.3.3 Pre-Installation Testing Requirements

Due to the potential for rough handling during shipment, all instrumentation shall be checked to ascertain that the equipment is in working order prior to installation. The pre-installation testing shall include a verification of the calibration data provided by the manufacturer, by checking two or three data points within the instrument measurement range. The verification testing shall be performed at a range of temperatures. Tests at the extreme temperature limits of the instrumentation may reveal malfunctions that could lead to erroneous data if not corrected. The pre-installation testing may consist of testing to determine if the instrumentation is in working order. This type of testing is also called function testing. Table 25-3 indicates some possible items for the pre-installation testing program.

Table 25-3, Possible Items in Pre-Installation Tests (Geotechnical Instrumentation – October 1998)

Category	Item
Data Supplied by Manufacturer	• Examine factory calibration curve and tabulated data to verify completeness • Examine manufacturer's final quality assurance inspection checklist, to verify completeness
Documentation	• Check, by comparing with procurement document, that model, dimensions, and materials are correct • Check that quantities received correspond to quantities ordered
Calibration Checks	• Check two or three points, if practicable • Check zero reading, e.g. of vibrating wire piezometers
Function Checks	• Connect to readout and induce change in parameter to be measured • Make and remake connectors several times, to verify correct functioning • Immerse in water, if applicable, and check
Electrical	• Perform resistance and insulation testing, in accordance with criteria provided by the instrument manufacturer
Mechanical	• Check cable length • Check tag numbers on instrument and cable • Verify all components fit together in the correct configuration • Check all components for signs of damage in transit

25.3.4 Calibration and Maintenance Requirements

Calibrations or function checks are required throughout the life of the instrumentation. Typically these calibrations are performed by the same personnel responsible for data collection. All calibrations and function checks shall be traceable (i.e., can be checked). The Contractor shall be required to develop a field calibration plan as part of the overall geotechnical instrumentation plan.

In addition to calibration, the personnel collecting the data shall also perform maintenance of the equipment. All maintenance shall be conducted in accordance with the manufacturer's requirements (if any is required).

25.3.5 Installation Methods

There are numerous ways to install the geotechnical instrumentation. The Special Provisions and Supplemental Specifications will provide some general requirements. The actual installation methods are left to the Contractor and shall be included in the installation plan. As part of the installation methods, the qualifications of the personnel installing the instrumentation shall also be included. The Contractor is solely responsible for installation and the performance of the instrumentation after installation. Badly performing or inoperative instrumentation shall be replaced at no additional cost to SCDOT.

25.3.6 Protection Plan

Geotechnical instrumentation that terminates at the ground surface (natural or man-made) is subject to damage by construction activities. Therefore, special precautions are required. As part of the installation plan, the Contractor is required to specify how the instrumentation is to be protected, not only from construction activities, but also from vandalism.

25.3.7 Installation Records

Detailed installation records are required to be submitted by the Contractor. These records fill two purposes. First, by requiring detailed installation records, the installation is more likely to be performed in accordance with accepted installation plan. Secondly, the records function as an "as-built" record and can indicate why the instrumentation is performing poorly or incorrectly, thus aiding the geotechnical engineer-of-record in determining if less reliance should be placed on a particular instrument. Having the record will also remove doubt if an instrument performs erratically by removing installation concerns as a potential cause of the problem. Presented in Table 25-4 are some items for possible inclusion on the installation record sheet.

Table 25-4, Possible Content of Installation Record Sheets (Geotechnical Instrumentation – October 1998)

Category	Content
Heading	• Project Name • Instrument type and number, including readout unit • Personnel responsible for installation • Date and time of start and completion
Planned Data	• Planned location in plan and elevation • Planned orientation • Planned lengths, widths, diameters, depths, and volumes of backfill • Necessary measurements or readings required during installation to ensure that all previous steps have been followed correctly, including post-installation acceptance tests
As-Built Data	• As-built location in plan and elevation • As-built orientation • As-built lengths, widths, diameters, depths, and volumes of backfill • Plant and equipment used, including diameter and depth of any drill casing used • A log of appropriate subsurface data • Type of backfill used • Post-Installation acceptance test
Weather	• Weather conditions
Notes	• Any notes, including problems encountered, delays, unusual features of the installation, and any events that may have a bearing on instrument behavior

25.3.8 Installation Reports

The purpose of the installation report is to provide a convenient summary of the information that personnel might need who are involved in the data collection, and processing, presentation and interpretation of the data. Listed below are some of the items that should be included in the report:

- Plans and sections sufficient to show instrument numbers and locations
- Appropriate surface and subsurface stratigraphic and geotechnical data
- Descriptions of instruments and readout units, including manufacturer's literature and photographs
- Details of calibration procedures
- Details of installation procedures (photographs are often helpful)
- Initial readings
- A copy of each installation record sheet

25.3.9 Data Collection Methods

Typically on SCDOT projects the collection of data is the responsibility of the Contractor, with the Contractor's personnel meeting the qualifications in the next Section. Data collection is typically obtained manually. In other words, physical measurements are made or the readout device is directly connected to the terminals of the instrument. Automatic Data Acquisition Systems (ADASs) are available; however, SCDOT does not have much experience in the use of these systems. Therefore, a manual collection system will be required if an ADAS is used. ADASs have the potential for remote downloading of the data, if the communications are properly setup.

25.3.10 Qualifications of Personnel Collecting Data

SCDOT requires that all personnel involved in the collection of instrument data be familiar with the instrumentation being used. These personnel shall be familiar with the installation report, so that if anomalies are encountered, they can provide feedback to the engineers processing the data. In addition, the personnel obtaining the data shall report to a licensed engineer. In the case of settlement plate readings, a licensed land surveyor is required. The qualifications of all personnel involved with the installation, calibration, maintenance and data collections shall be included as part of the Contractor's installation plan.

25.4 FIELD INSTRUMENTATION

The most commonly used types of field instrumentation for highway projects are discussed below. Included in the discussion are the role and typical uses of each instrument, a short description of methods commonly used, and common problems to be aware of with installation, reading, and interpretation of the instrumentation. For more information about particular instruments, the references cited at the end of the Chapter, as well as manufacturer manuals and websites, are recommended.

25.4.1 Slope Inclinometers

These instruments are used to monitor the magnitude, direction, and rate of subsurface deformations. Typical applications include monitoring the rate and extent of horizontal movement of embankments or cut slopes, determining the location of an existing failure surface, and monitoring deflection of retaining walls. Inclinometers can be installed at several levels on an embankment or cut slope to define the extent and nature of subsurface movements. An inclinometer consists of a grooved casing grouted vertically in a borehole. The role of the casing is to deform with the surrounding ground such that readings taken within the casing reflect accurate measurements of ground movement. Typically the grooves are aligned parallel to the direction of movement. The probe is periodically inserted down the casing and deflection of the casing is measured. The inclinometer probe contains accelerometers at either end to measure the parallel and perpendicular tilt of the casing. Successive measurements are plotted to provide a chronological indication of the extent and rate of subsurface movements.

Installation of inclinometer casing must be continued into rock or dense material that is not expected to deform. This will provide a point-of-fixity at the bottom of the casing to which other measurements through the casing can be reliably correlated to. Once drilling has proceeded to the desired depth and the inclinometer casing has been set in the borehole, the annulus between the casing and borehole side is filled with a grout that has a similar strength to that of the surrounding soil. Because the grout will induce a buoyant force on the casing, a stabilization method will be required to keep the casing in place during grout placement. Methods involving anchoring or weighting the casing bottom in the borehole are commonly used to overcome this issue. The instrument manufacturer should be consulted for recommended procedures for overcoming buoyancy. Holding the casing in place at the ground surface while grouting will cause the casing to corkscrew within the borehole which may cause errors in future readings. Inclinometers are to be installed and read in accordance with AASHTO Specification R 45-08 and the manufacturer's specifications.

25.4.2 <u>Settlement Monitoring</u>

These instruments are used to record the amount and rate of settlement under load. The most common installation of these instruments is for use with embankments where high settlements are predicted. The instruments listed below are the recommended methods for settlement measurement associated with highway embankments. Some instruments detailed below are designed to measure settlement through depth of strata. Because subsurface settlement instruments are often damaged during construction, some form of long-term settlement monitoring at the top of an embankment should be planned. This will provide a check of the readings obtained from subsurface instruments and can help to fill in the gaps from instruments that have either been damaged or have become unreliable.

25.4.2.1 Settlement Plate

The simplest form of settlement indicator is the settlement plate, which typically consists of a steel plate placed on the ground surface prior to embankment construction. The elevation of the initial plate must be recorded before construction begins to provide a reference point for all future readings. A reference rod and protective casing are then attached to the plate. As fill placement progresses, additional rods and casing are added. Settlement is measured by determining the elevation of the top of the reference rod at specified time intervals by surveying methods. The reference rod and initial platform elevations are determined relative to several benchmarks placed outside the construction area. Settlement plates are often placed in areas where the highest settlements are predicted.

25.4.2.2 Extensometers

The probe extensometer is another instrument commonly used to measure settlement. In a typical arrangement, corrugated polyethylene pipe surrounded by rings of stainless steel wire at selected intervals is lowered into a borehole. A rigid PVC inner pipe is coupled to the corrugated pipe prior to installation. Inclinometer casing is often used as the rigid inner pipe, thereby eliminating the need for drilling two separate boreholes for measuring horizontal and vertical displacement. The annulus between the rigid inner pipe and outer corrugated pipe is filled with bentonite slurry to minimize friction and the space between the outer pipe and borehole side is filled with a grout that conforms as nearly as possible to the properties of the surrounding soils. A more rigid system consisting of PVC pipe with telescopic couplings and steel plates instead of wire rings may be more desirable in situations where the likelihood of crushing the corrugated pipe exists, such as in high fill embankments or where high settlements are predicted.

The reading device in a probe extensometer consists of an induction coil housed within a probe attached to a signal cable that leads to a readout unit at the surface. As the probe is lowered, the operator notes at what depth the probe senses the steel rings, indicated by a buzzer on the readout unit. By comparing these depths to the initial depths, a settlement profile can be obtained. A main advantage of this type of instrument to a conventional settlement plate is that a settlement profile is obtained through the entire depth of the strata in question, not just at the surface. Optical surveying is typically not required so long as the bottom of the extensometer is fixed in stable ground. Drawbacks to this method include disruption to construction activities and cost, as compared to conventional settlement plates.

25.4.2.3 Settlement Cell

The settlement cell, or liquid-level gage instrument, consists of a pressure transducer embedded beneath the embankment with liquid-filled tubes connected to a reservoir and readout unit installed on stable ground. As the transducer settles, greater pressure is imparted on the transducer by the column of liquid. Settlement is measured by converting the increase in pressure to feet or meters of liquid head. This method requires that the liquid-filled tubes be run in trenches to areas outside of the construction area. Although trenching may cause some disruption to construction activity, all readings are taken away from the construction area after the instrument is installed. Settlement cells are often installed at several depths at the same cross-section to better define the full settlement profile. The ease of automation tends to be highest for this type of settlement measurement, especially if the pressure transducer is of the vibrating-wire type. A limitation to this type of instrument is that the soils surrounding the instrument and in the trench must be installed to specifications similar to that of the surrounding fill. Otherwise harmful discontinuities may be introduced into the embankment. This instrument should be used for short-term monitoring, because this instrument can be extremely temperature sensitive.

25.4.2.4 Settlement Reference Points

Settlement reference points are installed on structures or embankments upon essential completion of construction or topping out. Settlement reference points are intended to provide long-term settlement data by relatively simple methods at the ground surface. Settlement reference points may also be installed on embankments or structures such as a retaining wall to evaluate distress or unanticipated movement.

Settlement reference points are monitored using conventional surveying methods. Settlement reference points may consist of pins driven into the ground or mounted on a structure, or may simply be a painted reference point on a structure. Data collected over time indicates the amount of settlement that has occurred at each reference point. Care should be taken to protect settlement pins from disturbance by construction equipment or traffic that will affect the validity of data.

25.4.2.5 Crack Gauges

Crack gauges refer to simple commercial devices installed on a structure, such as a retaining wall, to visually monitor vertical and horizontal movements. Crack gauges permit visual monitoring and measurement of structural movements without requiring the use of survey equipment. Several configurations of the gauges are available, such as gauges mounted on a flat surface, or gauges mounted on either side of a corner.

Typical commercial crack gauges consist of two overlapping pieces of acrylic or PVC sheets fixed in place by epoxy. The sheets are installed so that the bottom sheet is fixed to the structure on one side of the crack, and the top sheet is fixed to the structure on the opposite side of the crack. The bottom sheet contains an opaque reference grid, and the top sheet is transparent with an intersecting vertical and horizontal marker. After measuring the width of the crack at the start of the monitoring period, horizontal and vertical movements of the structure can be monitored by noting the movement of the marker over the reference grid.

Crack gauges have some limitations and their use requires judgment and experience. Movements indicated on the gauge facing do not necessarily reflect the true peak movement which may occur in a dimension not recognized by an individual gauge as mounted. Crack gauges are typically only capable of monitoring movement in two dimensions; therefore, multiple gauges mounted at several locations on the structure will be required to monitor movement in three dimensions. When movements exceed the size of the reference grid, the size of the crack is recorded and new gauges can be installed to continue the monitoring program.

25.4.3 <u>Piezometers</u>

Piezometer applications generally fall into two categories: 1) Monitoring the flow of groundwater, or 2) Providing an index of soil strength. For highway construction, piezometers are typically installed to monitor pore water pressures associated with fill embankments and existing or cut slopes. Pore water pressure monitoring provides an estimate of effective stress within a slope. An increase in pore pressure indicated by a piezometer in a slope can be a signal of an impending slide. If a dewatering system is installed to stabilize a large excavation, piezometers can be used to gauge the effectiveness of the system. The most common use of piezometers in highway construction is to monitor the initial pore pressure rise and subsequent dissipation associated with consolidation of soils beneath an embankment. Pore pressure readings taken during construction of an embankment can be used to verify design settlement assumptions and to guide further construction activities.

The term piezometer is generally used to describe pore pressure monitoring instruments where seals are placed within the ground at selected depths, so as to monitor pore pressure conditions only within a certain strata. A device that has no seals is generally termed an observation well and should only be used in homogenous and continuously permeable soils. The simplest type of piezometer is an open standpipe piezometer. In this application, a section of slotted pipe attached to riser pipe is lowered to the desired elevation. A filter is generally placed around the slotted pipe and sand is placed in the borehole around the filter to create a reading interval. A bentonite seal is then placed atop the sand and a sealing grout is used to fill the remainder of the borehole. Open standpipe piezometers have a slower response time than some of the more sophisticated instruments described below, but are generally more cost effective to install and are more reliable than other methods.

Vibrating-wire piezometers are often used in applications where fast response to pore pressure changes is desired. Other advantages include less disruption to construction activity, less chance for damage in active construction zones (provided the lead cables are protected properly), and ease of reading and automation. A vibrating-wire piezometer consists of a diaphragm connected to a tensioned wire such that changes in pore-pressure affect the tension of the wire. A readout unit is used to pluck the wire and measure the change in wire tension, which can then be converted to pore-pressure readings. Vibrating-wire piezometers are typically installed in similar fashion to open-standpipe piezometers with the pressure transducer placed inside the screened reading interval, although recent research suggests that similar results can be obtained in a fully-grouted borehole. Please refer to the reference cited at the end of the Chapter for more information on the fully-grouted installation method. Push-in type vibrating-wire piezometers provide a quick and relatively easy installation and are commonly used to monitor pore pressure changes in successive lifts of an embankment. Open standpipe piezometers can also be converted to vibrating-wire piezometers simply by lowering a pressure transducer into the well to a specified depth. Most vibrating-wire type instruments currently come with some form of lightning protection housed inside the body of the instrument, though additional measures may be needed in areas prone to lightning activity.

Another piezometer type commonly used is the pneumatic piezometer, which consists of a flexible diaphragm and sensor body connected to a junction box at the surface with twin tubes. A filter is commonly used to separate the diaphragm from the surrounding material. Pressurized gas is introduced through the inlet tube. As gas pressure exceeds the pore water pressure, the diaphragm deflects, allowing gas to vent through the outlet tube. When the operator observes a return flow of gas, the gas supply is shut off and the diaphragm returns to its equilibrium position with the pore water pressure. The operator then obtains a reading from a pressure gauge connected to the input tube. This type of instrument also features a relatively short time lag and minimal disruption to construction. Some limitations of this instrument include the complexity of choosing the proper details of instrument, difficulty of reading, and the possibility of minute gas leaks within the system causing errors in data.

Oftentimes, it is not immediately known which type of piezometer is better suited to a particular application. One way of narrowing the choice and alleviating concerns over data reliability is to install groups of redundant piezometers of different types at similar locations and depths. Generally, open standpipe piezometers are paired with vibrating-wire or pneumatic piezometers and the data are periodically compared to ensure data validity. This setup also ensures that the flow of data will not be disrupted if one instrument malfunctions.

25.4.4 Tiltmeters

Tiltmeters are used to monitor the change in inclination from the norm of points on the ground or on structures. Typical highway applications include monitoring the tilt of MSE or conventional retaining walls and bridge columns. The complexity of tiltmeters can range from relatively simple instruments based on a plumb line or bubble level, to more sophisticated devices equipped with accelerometers to measure inclination housed inside a protective cover. Two common transducer types are servo-accelerometers and pendulum and vibrating-wire setups. Tiltmeters can either be permanently affixed to a structure or be portable. For the portable versions, a reference plate is attached to the structure and the portable instrument takes readings after it is attached to the plate in a repeatable position. The portable tiltmeter can be used to measure tilt biaxially by rotating the instrument 180 degrees on the reference plate and taking another reading. Fixed tiltmeters can also be used biaxially by installing two transducers on the same bracket at 90-degree angles to one another.

It is imperative that the tiltmeter reference plate or mounting bracket is attached securely to the structure that is to be monitored. They are typically cemented or screwed into place. A limitation of tilt measurements is that they tend to be more localized than with other types of field instrumentation. Extrapolating tilt measurements across a structure involves assumptions about the rigidity of the structure and therefore can be very difficult. For this reason, tiltmeters are generally used in conjunction with other deformation measurement methods such as inclinometers or surveying points.

25.4.5 Vibration Monitoring

Vibration-producing activities such as blasting, pile driving, vibratory compaction, and operation of other heavy equipment are common activities on a highway construction project. It may be desirable to monitor the ground vibrations induced by these activities if sensitive equipment or structures are located close to the work zone. Before starting work in an area that may impact nearby structures or equipment, a condition survey shall be performed to establish base line conditions and identify any pre-existing deficiencies. A thorough post-construction condition survey should also be performed to ensure that no structures have been harmed.

A vibration-monitoring unit generally consists of some combination of geophones, sound sensors and connecting cables attached to an input and readout unit. Ground vibrations are typically reported in terms of the peak particle velocity (ppv), although other parameters such as peak acceleration, principle frequencies, and peak sound pressure levels can also be obtained with most monitoring units. Vibration monitoring results are then compared with pre-established threshold levels of structures or equipment to determine the level of risk involved.

Vibrations associated with construction activity are generally monitored at specified distance intervals away from the source to at least as far as what is perceptible to the unit operator. This distance typically varies from 100 feet to over 600 feet based on the level of anticipated vibration at the source and location of nearby structures of interest. For the initial data, readings to a distance of at least 300 feet are recommended. Vibration monitoring units should be capable of detecting velocities of 0.1 in/sec. or less.

25.4.6 Special Instrumentation

Situations may arise where field instruments other than those described above are desired for use on a project. Many instruments, such as earth pressure cells or strain gauges, are typically not used in construction projects but only in research and special projects. Other instruments, such as borehole extensometers for monitoring a rock slope or tie-backs, may serve a key role on a project. Less common methods, such as horizontal inclinometers or other specialized instruments, should only be specified in special circumstances and with prior approval from appropriate SCDOT personnel. The need for special instrumentation and the selection of instruments will be evaluated on a case-by-case basis.

25.5 CONCLUSIONS

After assuring its validity, data from field instruments shall be interpreted relative to other instrument data as well as outside factors that may affect the data. For example, during construction of an embankment on soft ground, pore pressure rises and subsequent drops can be correlated to settlement measurements as well as the level of fill placement. A measured change in one instrument but not in other corresponding instruments may signal error stemming from either the instrument itself or reading methods. Another effective way to validate instrument readings is through routine visual observation. Observation of the monitored area can provide early warning signals, such as a tension crack or evident seepage, which may not be picked up by nearby field instruments and can also guide remedial actions.

The monitoring program of a highway construction project must be able to adapt to changing conditions. Base line readings of installed instruments may paint a picture that is totally different from what was assumed during the design phase. Components such as reading interval, methods of collecting data, and presentation of data may change dramatically over the course of a project.

25.6 REFERENCES

Dunnicliff, J. (1993), Geotechnical Instrumentation for Monitoring Field Performance, John Wiley, New York.

Dunnicliff, J. (1998), Geotechnical Instrumentation, FHWA Publication No. HI-98-034, US Department of Transportation, Federal Highway Administration, Office of Bridge Technology, Washington, DC.

Dunnicliff, J., Mikkelson, P.E., (2000), “Overcoming Buoyancy during Installation of Inclinometer Casing”, Geotechnical News, Vol. 18, No. 4, p. 23.

Mikkelson, P.E., Green, Gordon E. (2003), “Piezometers in Fully Grouted Boreholes”, FMGM 2003 - Field Measurements in Geomechanics, Oslo, Norway.

Chapter 26

GEOTECHNICAL SOFTWARE

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

Table of Contents

CHAPTER 26

GEOTECHNICAL SOFTWARE

26.1 INTRODUCTION

This Chapter provides a general overview of software used in geotechnical engineering and analysis. Software is used in geotechnical engineering to speed computations and to perform complex analyses. There are two types of software used by SCDOT: 1) software packages developed and marketed by university and software development firms, 2) spreadsheets and customized software packages developed by individuals or companies. Typically the first type of software is commercially available and used by multiple government agencies and private entities. The second type typically is only used by a single agency or entity and developed locally by the user. SCDOT recognizes that both types of software are used; however, the individually developed software requires QA/QC verification prior to being used.

Prior to the use of any software, it is incumbent upon the engineer to understand how the software determines the results and any limitations that are inherent to the software. For example, the SHAKE program becomes unreliable if the strains exceed three percent (3%). SCDOT recommends that prior to using a new software program, the user should run at least one example and check the results against a set of hand calculations.

Appendix G provides a list of software that is currently used by SCDOT. Consultants are not required to obtain the same software as SCDOT, but all software shall use the models permitted in this Manual.

26.2 COMMERICAL SOFTWARE

As indicated previously, commercially available software packages are typically produced by either a university or software development firm and sold for profit. Software developed in this manner normally goes through extensive QA/QC prior to being sold. Therefore, the only documentation SCDOT requires for these software packages is the contact information for the developer. Software obtained from the FHWA website requires no additional information.

26.3 NON-COMMERICAL SOFTWARE

Non-commercial software packages are those packages that are developed by and used internally in a single agency or entity. Included in this category are spreadsheets, MathCAD programs or software developed using computer language, such as FORTRAN, C++ or similar. Because these types of packages may not get the same level of review as commercially developed software, additional information and supporting documentation may be required prior to their acceptance by SCDOT. The supporting documentation may include, but is not limited to, an example prepared using the software and a set of hand calculations for the same example. In addition to the example, a QA/QC plan for the development of software packages shall also be prepared, indicating the person or persons who reviewed the software. For software developed internally by the Department, the software should be reviewed by the PCS/GDS.

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	Jeff Sizemore, P.E.
Geotechnical Professional	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:

File No. _______, PIN __________
____
____
_________________ County

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

Appendix B

SLOPE STABILITY DESIGN CHARTS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

APPENDIX B

SLOPE STABILITY DESIGN CHARTS

Appendix B – Slope Stability Design Charts contains design charts obtained from Slope Stability, Appendix E – Chart Solutions for Embankment Slopes, Appendix A – References and Appendix B - Notation. Slope Stability is published by the US Army Corps of Engineers, EM 1110-2-1902, dated October 31, 2003. These Appendices are used with permission of the US Army Corps of Engineers. SCDOT assumes no responsibility for any errors that may be found with this Appendix.

EM 1110-2-1902
31 Oct 2003

US Army Corps
of Engineers®

ENGINEERING AND DESIGN

Slope Stability

ENGINEER MANUAL

Appendix E
Chart Solutions for Embankment Slopes

E-1. General Use and Applicability of Slope Stability Charts

a. Slope stability charts provide a means for rapid analysis of slope stability. They can be used for preliminary analyses, for checking detailed analyses, or for complete analyses. They are especially useful for making comparisons between design alternatives, because they provide answers so quickly. The accuracy of slope stability charts is usually as good as the accuracy with which shear strengths can be evaluated.

b. In this appendix, chart solutions are presented for four types of slopes:

(1) Slopes in soils with $\phi = 0$ and uniform strength throughout the depth of the soil layer.

(2) Slopes in soils with $\phi > 0$ and $c > 0$ and uniform strength throughout the depth of the soil layer.

(3) Infinite slopes in soils with $\phi > 0$ and $c = 0$ and soils with and $\phi > 0$ and $c > 0$.

(4) Slopes in soils with $\phi = 0$ and strength increasing linearly with depth.

Using approximations in slope geometry and carefully selected soil properties, these chart solutions can be applied to a wide range of nonhomogenous slopes.

c. This appendix contains the following slope stability charts:

- Figure E-1: Slope stability charts for $\phi = 0$ soils (after Janbu 1968).[1]
- Figure E-2: Surcharge adjustment factors for $\phi = 0$ and $\phi > 0$ soils (after Janbu 1968).
- Figure E-3: Submergence and seepage adjustment factors for $\phi = 0$ and $\phi > 0$ soils (after Janbu 1968).
- Figure E-4: Tension crack adjustment factors for $\phi = 0$ and $\phi > 0$ soils (after Janbu 1968).
- Figure E-5: Slope stability charts for $\phi > 0$ soils (after Janbu 1968).
- Figure E-6: Steady seepage adjustment factor for $\phi > 0$ soils (after Duncan, Buchianani, and DeWet 1987).
- Figure E-7: Slope stability charts for infinite slopes (after Duncan, Buchianani, and DeWet 1987).
- Figure E-8: Slope stability charts for $\phi = 0$ soils, with strength increasing with depth (after Hunter and Schuster 1968).

E-2. Averaging Slope Inclinations, Unit Weights and Shear Strengths

a. For simplicity, charts are developed for simple homogenous soil conditions. To apply them to nonhomogeneous conditions, it is necessary to approximate the real conditions with an equivalent homogenous slope. The most effective method of developing a simple slope profile for chart analysis is to begin with a cross section of the slope drawn to scale. On this cross section, using judgment, draw a geometrically simple slope that approximates the real slope as closely as possible.

---

[1] Reference information is presented in Appendix A.

b. To average the shear strengths for chart analysis, it is useful to know the location of the critical slip surface. The charts contained in the following sections of this appendix provide a means of estimating the position of the critical circle. Average strength values are calculated by drawing the critical circle, determined from the charts, on the slope. Then the central angle of arc subtended within each layer or zone of soil is measured with a protractor. The central angles are used as weighting factors to calculate weighted average strength parameters, c_{avg} and ϕ_{avg} are as follows:

$$c_{avg} = \frac{\sum \delta_i c_i}{\sum \delta_i} \tag{E-1}$$

$$\phi_{avg} = \frac{\sum \delta_i \phi_i}{\sum \delta_i} \tag{E-2}$$

where

c_{avg} = average cohesion (stress units)

ϕ_{avg} = average angle of internal friction (degrees)

δ_i = central angle of arc, measured around the center of the estimated critical circle, within zone i (degrees)

c_i = cohesion in zone i (stress units)

ϕ_i = angle of internal friction in zone i

c. One condition in which it is preferable not to use these averaging procedures is the case in which an embankment overlies a weak foundation of saturated clay, with $\phi = 0$. Using Equations E-1 and E-2 to develop average values of c and ϕ in such a case would lead to a small value of ϕ_{avg} (perhaps 2 to 5 degrees). With $\phi_{avg} > 0$, it would be necessary to use the chart shown in Figure E-5, which is based entirely on circles that pass through the toe of the slope. With $\phi = 0$ foundation soils, the critical circle usually goes below the toe into the foundation. In these cases, it is better to approximate the embankment as a $\phi = 0$ soil and to use the $\phi = 0$ slope stability charts shown in Figure E-1. The equivalent $\phi = 0$ strength of the embankment soil can be estimated by calculating the average normal stress on the part of the slip surface within the embankment (one-half the average vertical stress is usually a reasonable approximation of the normal stress on this part of the slip surface) and determining the corresponding shear strength at that point on the shear strength envelope for the embankment soil. This value of strength is treated as a value of S_u for the embankment, with $\phi = 0$. The average value of S_u is then calculated for both the embankment and the foundation using the same averaging procedure as described above.

$$(S_u)_{avg} = \frac{\sum \delta_i (S_u)_i}{\sum \delta_i} \tag{E-3}$$

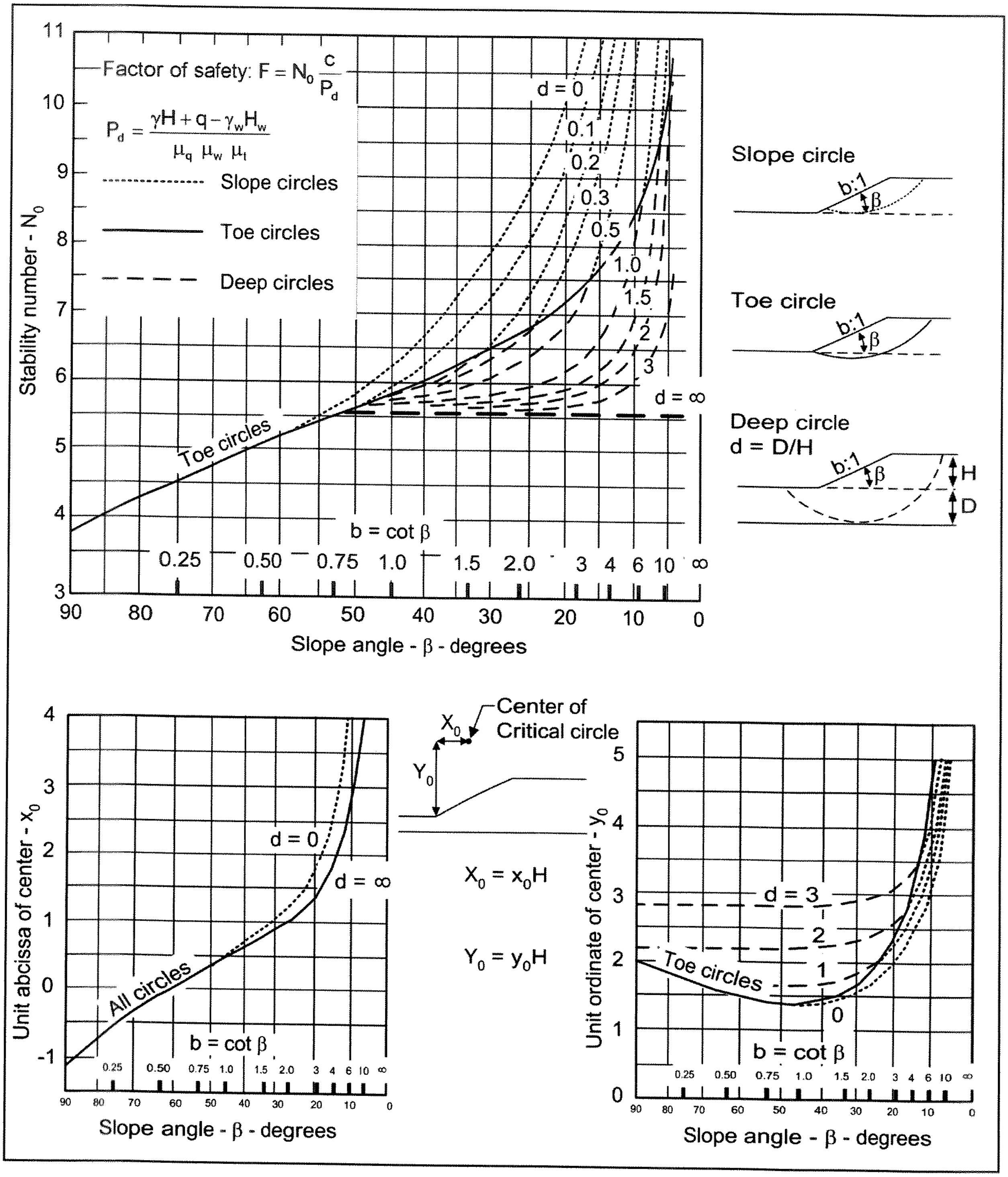


Figure E-1. Slope stability charts for $\phi = 0$ soils (after Janbu 1968)

where

$(S_u)_{avg}$ = average undrained shear strength (in stress units)

$(S_u)_i$ = S_u in layer i (in stress units)

δ_i = central angle of arc, measured around the center of the estimated critical circle, within zone i (degrees)

This average value of S_u is then used, with $\phi = 0$, for analysis of the slope.

d. To average unit weights for use in chart analysis, it is usually sufficient to use layer thickness as a weighting factor, as indicated by the following expression:

$$\gamma_{avg} = \frac{\sum \gamma_i h_i}{\sum h_i} \tag{E-4}$$

where

γ_{avg} = average unit weight (force per length cubed)

γ_i = unit weight of layer i (force per length cubed)

h_i = thickness of layer i (in length units)

e. Unit weights should be averaged only to the depth of the bottom of the critical circle. If the material below the toe of the slope is a $\phi = 0$ material, the unit weight should be averaged only down to the toe of the slope, since the unit weight of the material below the toe has no effect on stability in this case.

E-3. Soils with ϕ = 0

The slope stability chart for $\phi = 0$ soils, developed by Janbu (1968), is shown in Figure E-1. Charts providing adjustment factors for surcharge loading at the top of the slope are shown in Figure E-2. Charts providing adjustment factors for submergence and seepage are shown in Figure E-3. Charts providing adjustment factors to account for tension cracks are shown in Figure E-4.

a. Steps for use of $\phi = 0$ charts:

(1) Using judgment, decide which cases should be investigated. For uniform soil conditions, the critical circle passes through the toe of the slope if the slope is steeper than about 1 (H) on 1 (V). For flatter slopes, the critical circle usually extends below the toe, and is tangent to some deep firm layer. The chart in Figure E-1 can be used to compute factors of safety for circles extending to any depth. Multiple possibilities should be analyzed, to be sure that the overall critical circle and overall minimum factor of safety have been found.

(2) The following criteria can be used to determine which possibilities should be examined:

(a) If there is water outside the slope, a circle passing above the water may be critical.

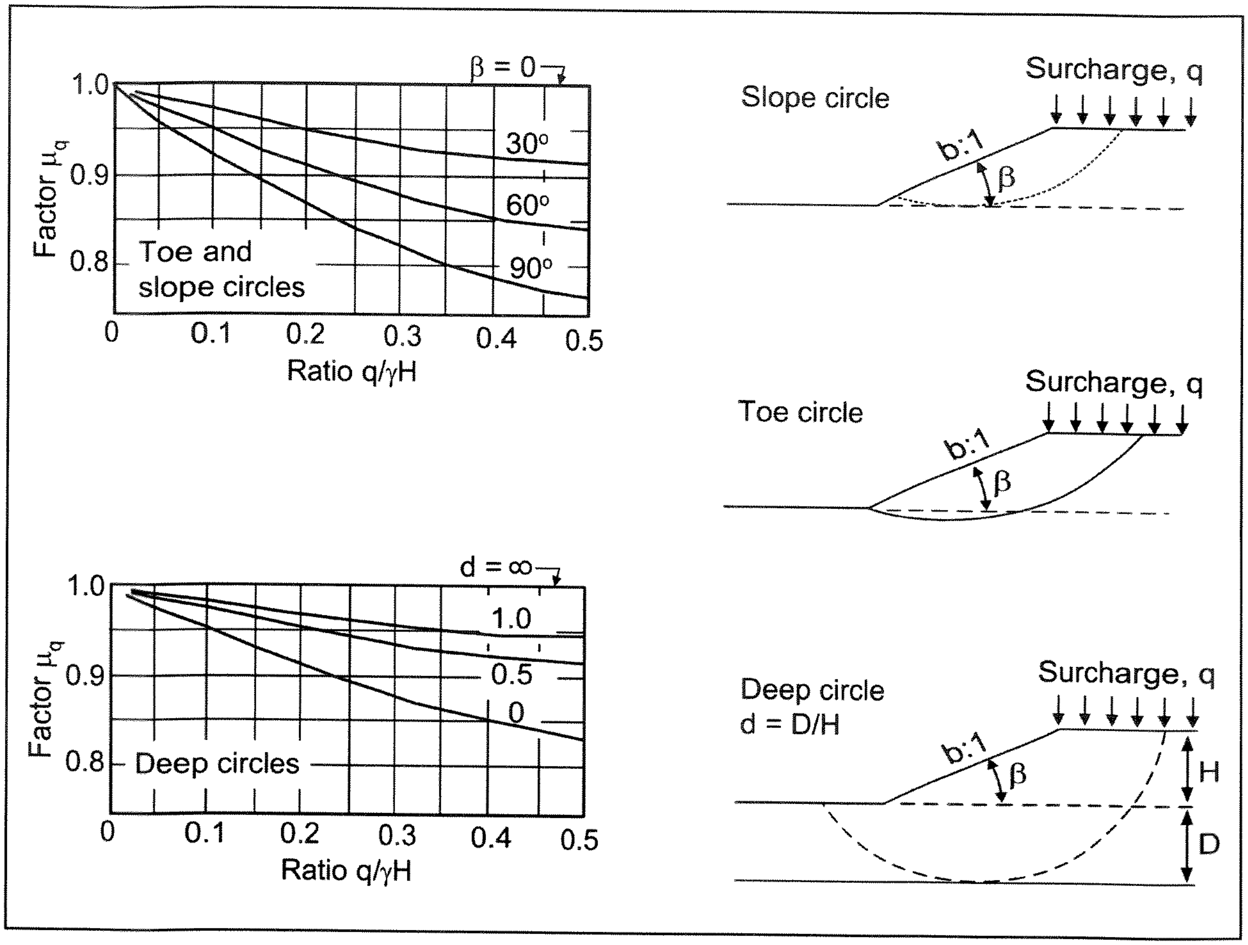


Figure E-2. Surcharge adjustment factors $\phi = 0$ and $\phi > 0$ soils (after Janbu 1968)

(b) If a soil layer is weaker than the one above it, the critical circle may be tangent to the base of the lower (weaker) layer. This applies to layers both above and below the toe.

(c) If a soil layer is stronger than the one above it, the critical circle may be tangent to the base of either layer, and both possibilities should be examined. This applies to layers both above and below the toe.

(3) The following steps are performed for each circle:

(a) Calculate the depth factor, d, using the formula:

$$d = \frac{D}{H} \tag{E-5}$$

where

D = depth from the toe of the slope to the lowest point on the slip circle (L; length)

H = slope height above the toe of the slope (L)

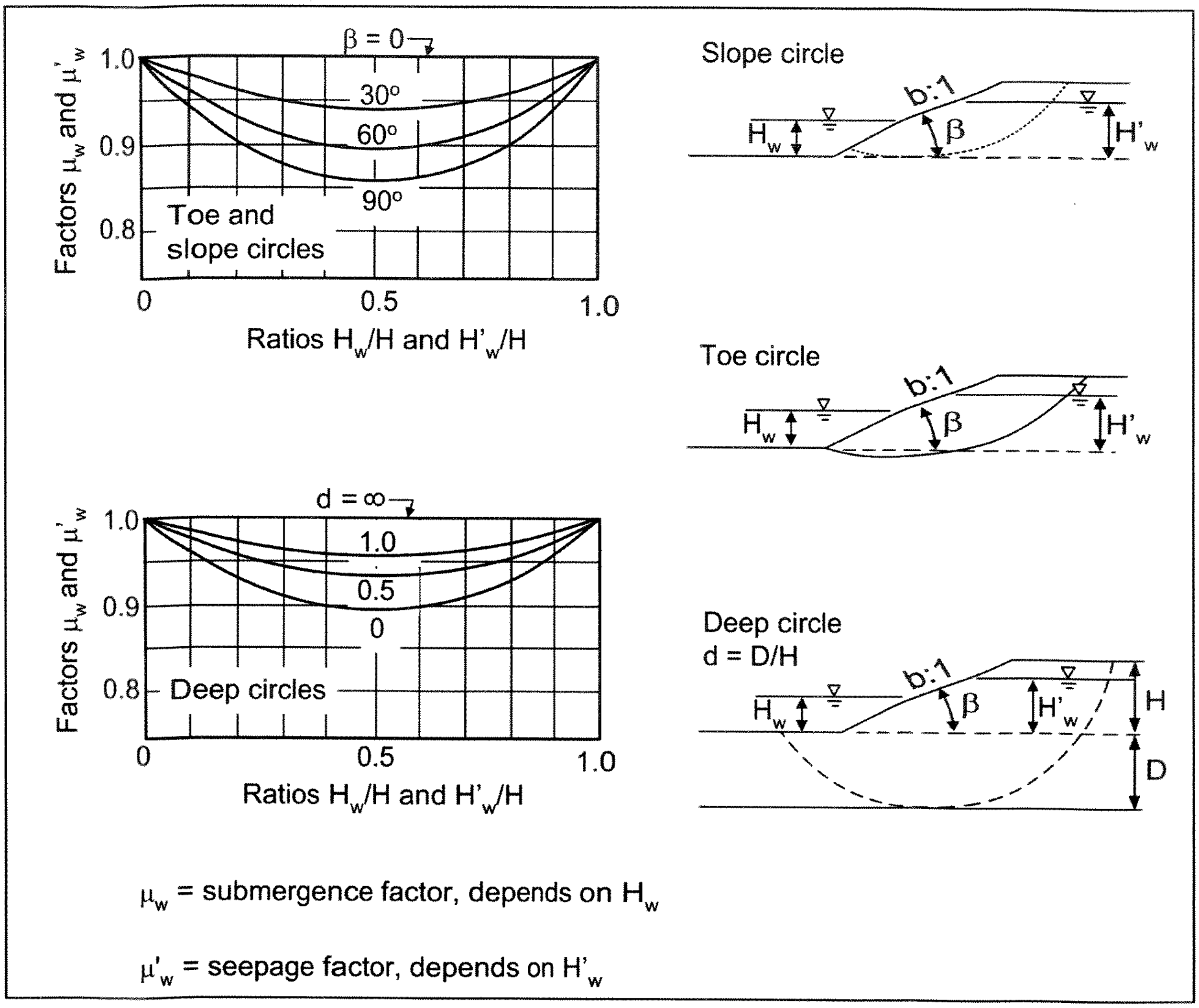


Figure E-3. Submergence and seepage adjustment factors $\phi = 0$ and $\phi > 0$ soils (after Janbu 1968)

The value of d is 0 if the circle does not pass below the toe of the slope. If the circle being analyzed is entirely above the toe, its point of interaction with the slope should be taken as an "adjusted toe," and all dimensions like D, H, and H_w must be adjusted accordingly in the calculations.

(b) Find the center of the critical circle using the charts at the bottom of Figure E-1, and draw this circle to scale on a cross section of the slope.

(c) Determine the average value of the strength, $c = S_u$, for the circle, using Equation E-3.

(d) Calculate the quantity P_d using Equation E-6:

$$P_d = \frac{\gamma H + q - \gamma_w H_w}{\mu_q \mu_w \mu_t} \qquad \text{(E-6)}$$

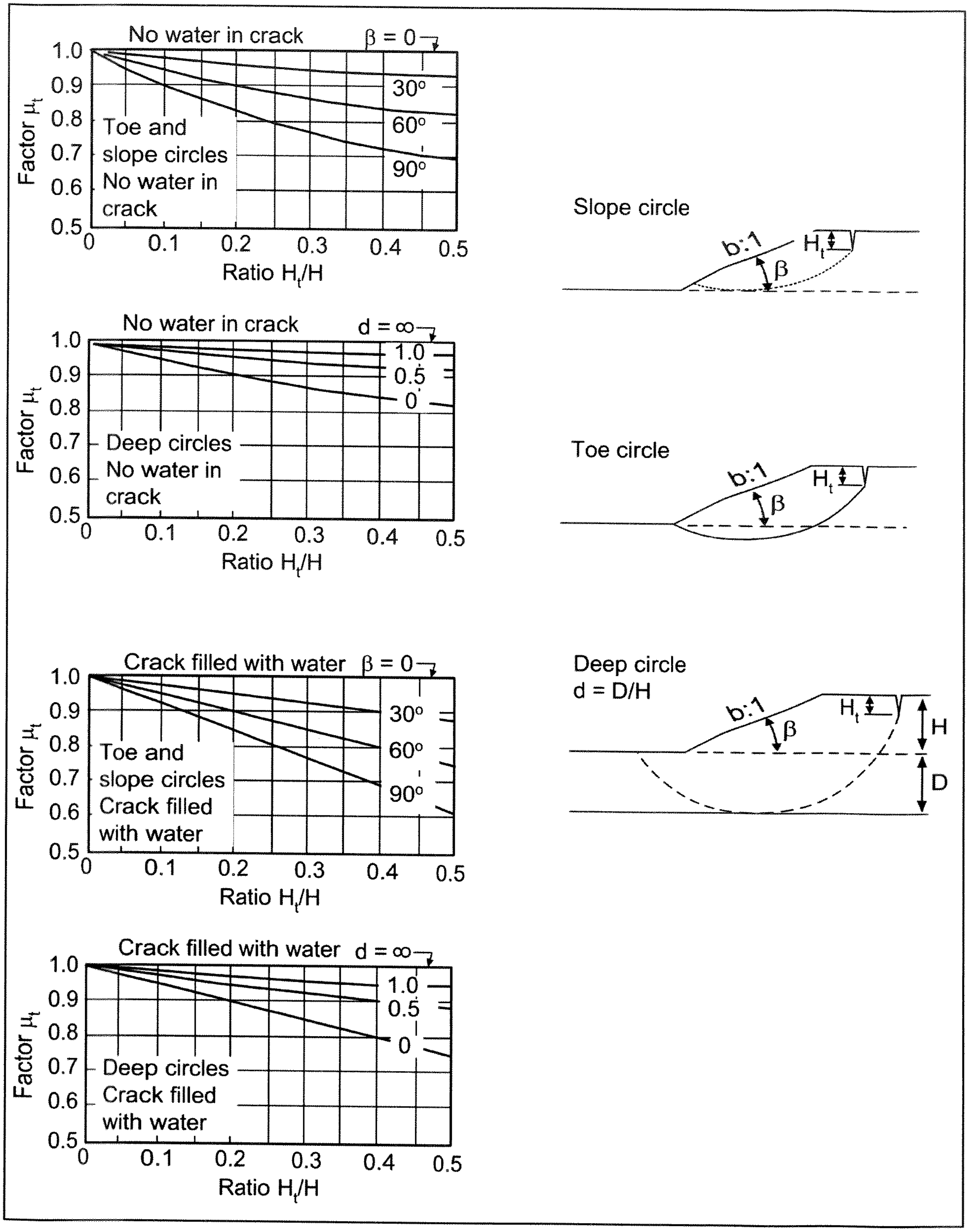


Figure E-4. Tension crack adjustment factors $\phi = 0$ and $\phi > 0$ soils (after Janbu 1968)

where

γ = average unit weight of soil (F/L^3))

H = slope height above toe (L)

q = surcharge (F/L^2)

γ_w = unit weight of water (F/L^3)

H_w = height of external water level above toe (L)

μ_q = surcharge adjustment factor (Figure E-2)

μ_w = submergence adjustment factor (Figure E-3)

μ_t = tension crack adjustment factor (Figure E-4)

If there is no surcharge, $\mu_q = 1$.

If there is no external water above toe, $\mu_w = 1$; and if there are no tension cracks, $\mu_t = 1$.

(e) Using the chart at the top of Figure E-1, determine the value of the stability number, N_o, which depends on the slope angle, β, and the value of d.

(f) Calculate the factor of safety, F, using Equation E-7:

$$F = \frac{N_o c}{P_d} \tag{E-7}$$

where

N_o = stability number

c = average shear strength = $(S_u)_{avg}$ (F/L^2)

b. The example problems in Figures E-9 and E-10 illustrate the use of these methods. Note that both problems involve the same slope, and that the only difference between the two problems is the depth of the circle analyzed.

E-4. Soils with $\phi > 0$

a. The slope stability chart for $\phi > 0$ soils, developed by Janbu (1968), is shown in Figure E-5.

b. Adjustment factors for surcharge are shown in Figure E-2. Adjustment factors for submergence and seepage are shown in Figure E-3. Adjustment factors for tension cracks are shown in Figure E-4.

c. The stability chart in Figure E-5 can be used for analyses in terms of effective stresses. The chart may also be used for total stress analysis of unsaturated slopes with $\phi > 0$.

d. Steps for use of charts are:

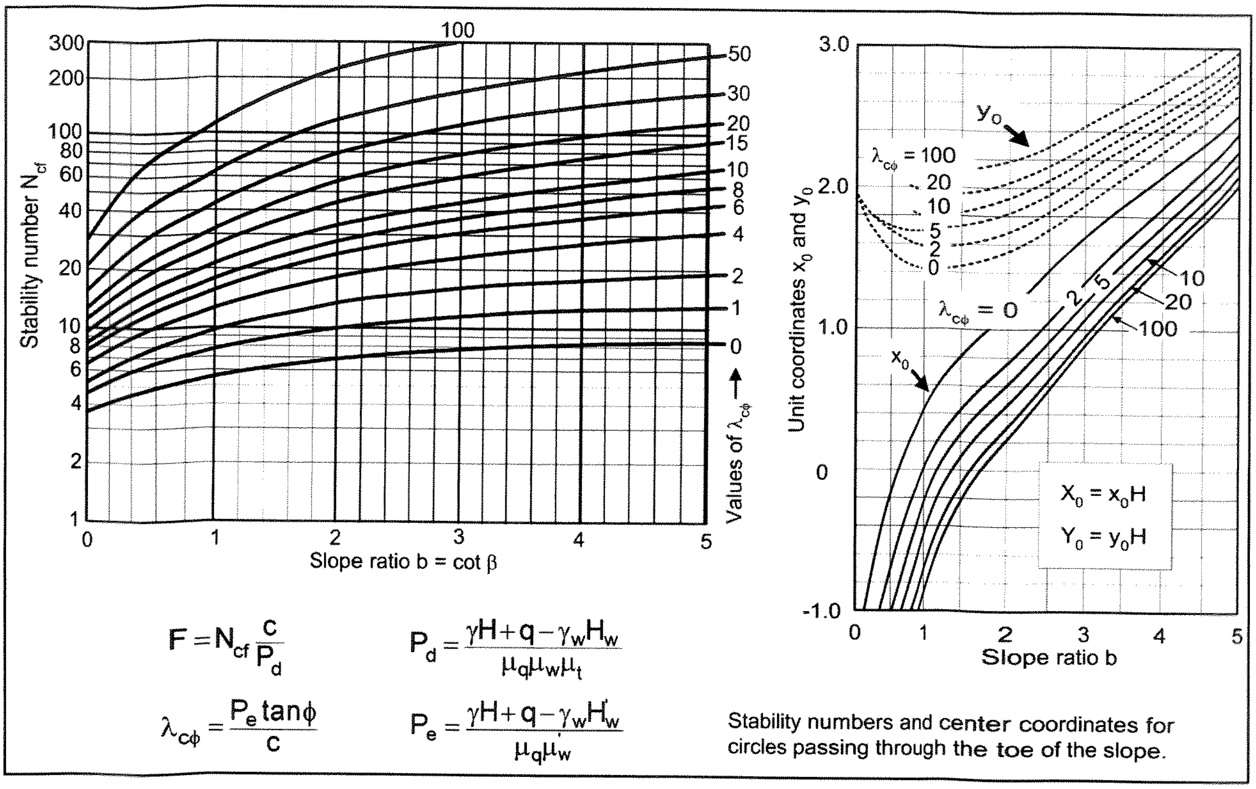


Figure E-5. Slope stability charts for $\phi > 0$ soils (after Janbu 1968)

(1) Estimate the location of the critical circle. For most conditions of slopes in uniform soils with $\phi > 0$, the critical circle passes through the toe of the slope. The stability numbers given in Figure E-5 were developed by analyzing toe circles. In cases where c = 0, the critical mechanism is shallow sliding, which can be analyzed as the infinite slope failure mechanism. The stability chart shown in Figure E-7 can be used in this case. If there is water outside the slope, the critical circle may pass above the water. If conditions are not homogeneous, a circle passing above or below the toe may be more critical than the toe circle. The following criteria can be used to determine which possibilities should be examined:

(a) If there is water outside the slope, a circle passing above the water may be critical.

(b) If a soil layer is weaker than the one above it, the critical circle may be tangent to the base of the lower (weaker) layer. This applies to layers both above and below the toe.

(c) If a soil layer is stronger than the one above it, the critical circle may be tangent to the base of either layer, and both possibilities should be examined. This applies to layers both above and below the toe.

The charts in Figure E-5 can be used for nonuniform conditions provided the values of c and ϕ used in the calculation represent average values for the circle considered. The following steps are performed for each circle.

(2) Calculate P_d using the formula:

$$P_d = \frac{\gamma H + q - \gamma_w H_w}{\mu_q \mu_w \mu_t} \tag{E-8}$$

where

γ = average unit weight of soil (F/L^3))

H = σλοπε height above toe (L)

q = συρχηαργε (F/L^2)

γ_w = unit weight of water (F/L^3)

H_w = height οφ external water level above toe (L)

μ_q = surcharge reduction factor (Figure E-2)

μ_w = submergence reduction factor (Figure E-3)

μ_t = tension crack reduction factor (Figure E-4)

μ_q = 1, if there is no surcharge

μ_w = 1, if there is no external water above toe

μ_t = 1, if there are no tension cracks

If the circle being studied passes above the toe of the slope, the point where the circle intersects the slope face should be taken as the toe of the slope for the calculation of H and H_w.

(3) Calculate P_e using the formula:

$$P_e = \frac{\gamma H + q - \gamma_w H_w'}{\mu_q \mu_w'} \qquad \text{(E-9)}$$

where

H_w' = height of water within slope (L)

μ_w' = seepage correction factor (Figure E-3)

The other factors are as defined previously.

H_w' is the average level of the piezometric surface within the slope. For steady seepage conditions this is related to the position of the phreatic surface beneath the crest of the slope as shown in Figure E-6 (after Duncan, Buchignani, and DeWet 1987). If the circle being studied passes above the toe of the slope, H_w' is measured relative to the adjusted toe.

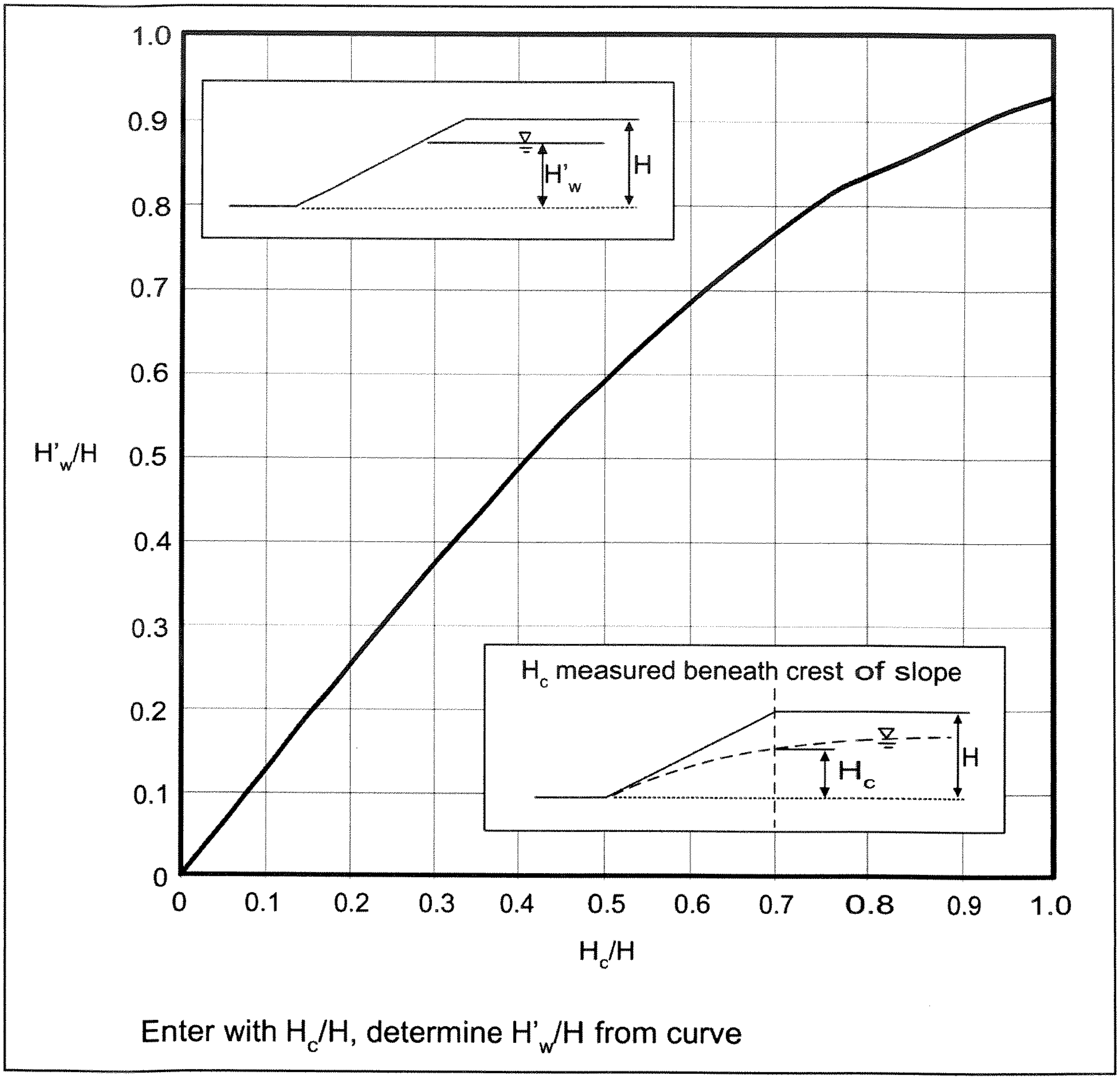


Figure E-6. Steady seepage adjustment factor ϕ > 0 soils (after Duncan, Buchianani, and DeWet 1987)

μ_w' = 1, if there is no seepage

μ_q = 1, if there is no surcharge

In a total stress analysis, internal pore water pressure is not considered, so $H_w' = 0$ and $\mu_w' = 1$ in the formula for P_e.

(4) Calculate the dimensionless parameter $\lambda_{c\phi}$ using the formula:

$$\lambda_{c\phi} = \frac{P_e \tan\phi}{c} \qquad \text{(E-10)}$$

where

ϕ = average value of ϕ

c = average value of c (F/L^2)

For c = 0, $\lambda_{c\phi}$ is infinite. Use the charts for infinite slopes in this case. Steps 4 and 5 are iterative steps. On the first iteration, average values of tan ϕ and c are estimated using judgment rather than averaging.

(5) Using the chart at the top of Figure E-5, determine the center coordinates of the circle being investigated.

(a) Plot the critical circle on a scaled cross section of the slope, and calculate the weighted average values of ϕ and c using Equations E-1 and E-2.

(b) Return to Step 4 with these average values of the shear strength parameters, and repeat this iterative process until the value of $\lambda_{c\phi}$ becomes constant. Usually one iteration is sufficient.

(6) Using the chart at the left side of Figure E-5, determine the value of the stability number N_{cf}, which depends on the slope angle, β, and the value of $\lambda_{c\phi}$.

(7) Calculate the factor of safety, F, using the formula:

$$F = N_{cf}\frac{c}{P_d} \qquad \text{(E-11)}$$

The example problems in Figures E-11 and E-12 illustrate the use of these methods for total stress and effective stress analyses.

E-5. Infinite Slope Analyses

a. Two types of conditions can be analyzed using the charts shown in Figure E-7: These are:

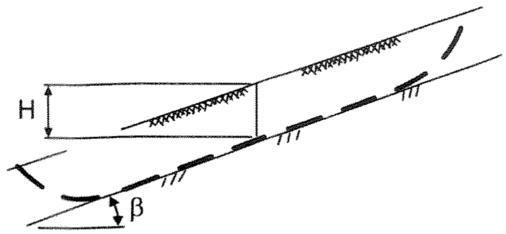
(1) Slopes in cohesionless materials, where the critical failure mechanism is shallow sliding or surface raveling.

(2) Slopes in residual soils, where a relatively thin layer of soil overlies firmer soil or rock, and the critical failure mechanism is sliding along a plane parallel to the slope, at the top of the firm layer.

b. Steps for use of the charts for effective stress analyses:

(1) Determine the pore pressure ratio, r_u, which is defined by the formula:

$$r_u = \frac{u}{\gamma H} \qquad \text{(E-12)}$$



γ = total unit weight of soil

γ_w = unit weight of water

c' = cohesion intercept

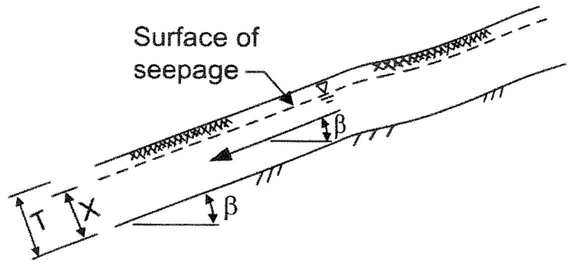
ϕ' = friction angle

r_u = pore pressure ratio = $u/\gamma H$

u = pore pressure at depth H

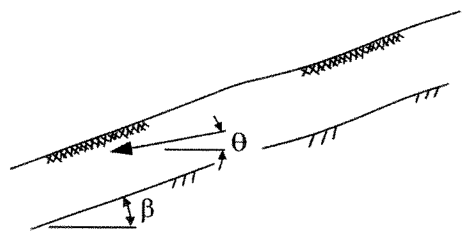
<u>Steps:</u>

1. Determine r_u from measured pore pressure or formulas at right.
2. Determine A and B from charts below.
3. Calculate $F = A\dfrac{\tan\phi'}{\tan\beta} + B\dfrac{c'}{\gamma H}$



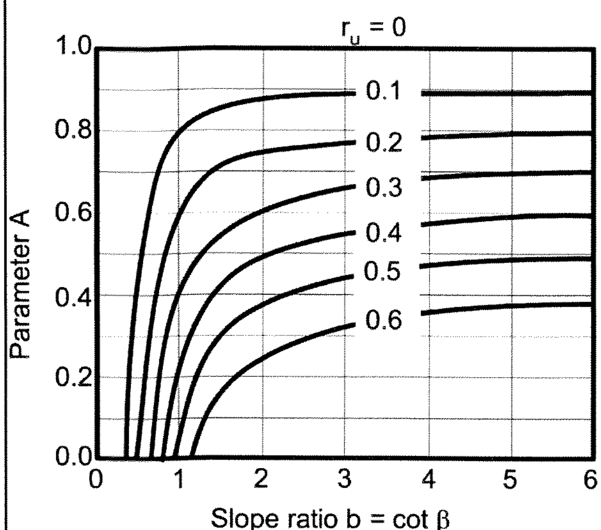
Seepage parallel to slope

$$r_u = \frac{X}{T}\frac{\gamma_w}{\gamma}\cos^2\beta$$



Seepage emerging from slope

$$r_u = \frac{\gamma_w}{\gamma}\frac{1}{1+\tan\beta\tan\theta}$$



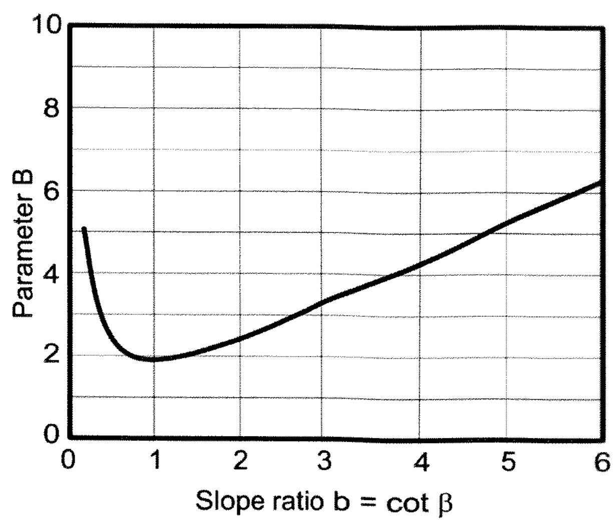


Figure E-7. Slope stability charts for infinite slopes (after Duncan, Buchianani, and DeWet 1987)

where

u = pore pressure (F/L^2)

γ = total unit weight of soil (F/L^3)

H = depth corresponding to pore pressure, u (L)

(a) For an existing slope, the pore pressure can be determined from field measurements, using piezometers installed at the depth of sliding, or estimated for the most adverse anticipated seepage condition.

(b) For seepage parallel to the slope, which is a condition frequently used for design, the value of r_u can be calculated using the following formula:

$$r_u = \frac{X}{T}\frac{\gamma_w}{\gamma}\cos^2\beta \qquad \text{(E-13)}$$

where

X = distance from the depth of sliding to the surface of seepage, measured normal to the surface of the slope (L)

T = distance from the depth of sliding to the surface of the slope, measured normal to the surface of the slope (L)

γ_w = unit weight of water (F/L^3)

γ = total unit weight of soil (F/L^3)

β = slope angle

(c) For seepage emerging from the slope, which is more critical than seepage parallel to the slope, the value or r_u can be calculated using the following formula:

$$r_u = \frac{\gamma_w}{\gamma}\frac{1}{1+\tan\beta\tan\theta} \qquad \text{(E-14)}$$

where

θ = angle of seepage measured from the horizontal direction

The other factors are as defined previously.

(1) Submerged slopes with no excess pore pressures can be analyzed using $\gamma = \gamma_b$ (buoyant unit weight) and $r_u = 0$.

(2) Determine the values of the dimensionless parameters A and B from the charts at the bottom of Figure E-7.

(3) Calculate the factor of safety, F, using Equation E-15:

$$F = A\frac{\tan\phi'}{\tan\beta} + B\frac{c'}{\gamma H} \tag{E-15}$$

where

ϕ' = angle of internal friction in terms of effective stress

c' = cohesion intercept in terms of effective stress (F/L^2)

β = slope angle

H = depth of sliding mass measured vertically (L)

The other factors are as defined previously.

c. Steps for use of charts for total stress analyses:

(1) Determine the value of B from the chart in the lower right corner of Figure E-7.

(2) Calculate the factor of safety, F, using the formula:

$$F = \frac{\tan\phi}{\tan\beta} + B\frac{c}{\gamma H} \tag{E-16}$$

where

ϕ = angle of internal friction in terms of total stress

c = cohesion intercept in terms of total stress (F/L^2)

The other factors are as defined previously.

The example in Figure E-13 illustrates use of the infinite slope stability charts.

E-6. Soils with $\phi = 0$ and Strength Increasing with Depth

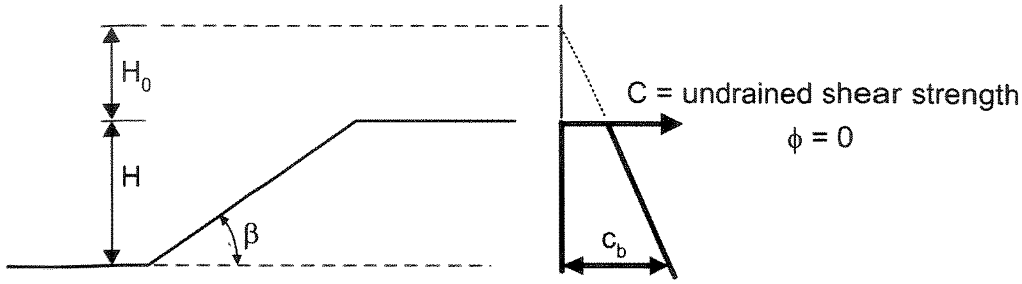
The chart for slopes in soils with $\phi = 0$ and strength increasing with depth is shown in Figure E-8. Steps for use of the chart are:

a. Select the linear variation of strength with depth that best fits the measured strength data. As shown in Figure E-8, extrapolate this straight line upward to determine H_o, the height at which the straight line intersects zero.

b. Calculate $M = H_o/H$, where H = slope height.

c. Determine the dimensionless stability number, N, from the chart in the lower right corner of Figure E-8.

d. Determine the value of c_b, the strength at the elevation of the bottom (the toe) of the slope.



Steps

1. Extrapolate strength profile upward to determine value of H_0, where strength profile intersects zero.
2. Calculate $M = H_0/H$.
3. Determine stability number from chart below.
4. Determine c_b = strength at elevation of toe of slope.
5. Calculate $F = N \dfrac{c_b}{\gamma(H + H_0)}$

Use $\gamma = \gamma_{buoyant}$ for submerged slope

Use $\gamma = \gamma_{total}$ for no water outside slope

Use average γ for partly submerged slope

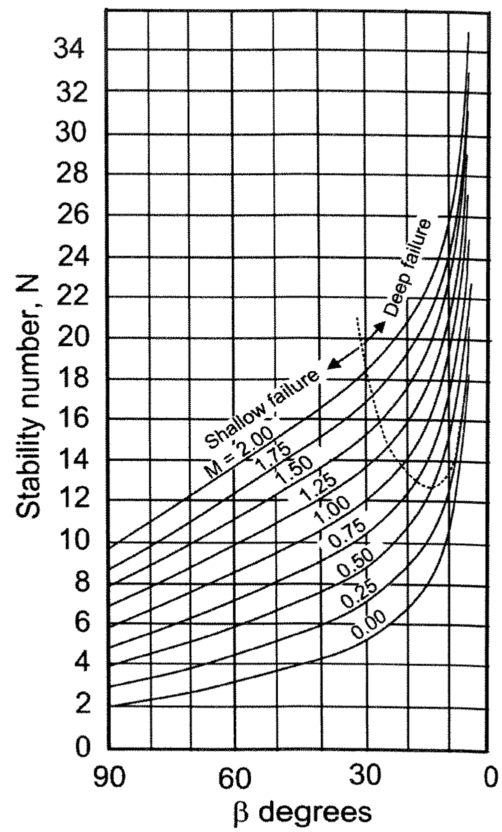


Figure E-8. Slope stability chart for $\phi = 0$ soils, with strength increasing with depth (after Hunter and Schuster 1968) *e.* Calculate the factor of safety, F, using the formula:

$$F = N\frac{c_b}{\gamma(H + H_o)} \tag{E-17}$$

where

γ = total unit weight of soil for slopes above water

γ = buoyant unit weight for submerged slopes

γ = weighted average unit weight for partly submerged slopes

The example shown in Figure E-14 illustrates use of the stability chart shown in Figure E-8.

Example Problem E-1

Figure E-9 shows a slope in $\phi = 0$ soil. There are three layers with different strengths. There is water outside the slope.

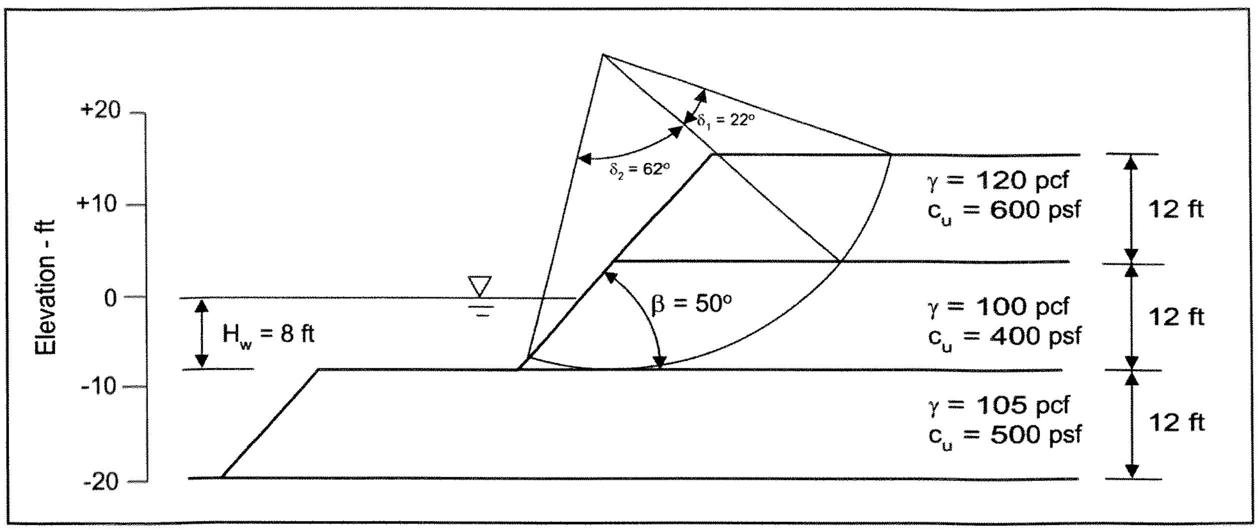


Figure E-9. Example for a circle tangent to elevation –8ft for cohesive soil with ϕ = 0

Two circles were analyzed for this slope -- a shallow circle tangent to elevation -8 ft, and a deep circle tangent to elevation -20 ft.

The shallower circle, tangent to elevation –8 ft, is analyzed first.

For this circle:

$$d = \frac{D}{H} = \frac{0}{24} = 0$$

$$\frac{H_w}{H} = \frac{8}{24} = 0.33$$

Using the charts at the top of Figure E-1, with $\beta = 50°$ and $d = 0$:

$x_o = 0.35$ and $y_o = 1.4$

$X_o = (H)(x_o) = (24)(0.35) = 8.4$ ft

$Y_o = (H)(y_o) = (24)(1.4) = 33.6$ ft

Plot the critical circle on the slope. The circle is shown in Figure E-9.

Measure the central angles of arc in each layer using a protractor. Calculate the weighted average strength parameter c_{avg} using Equation E-1.

$$c_{avg} = \frac{\sum \delta_i c_i}{\sum \delta_i} = \frac{(22)(600)+(62)(400)}{22+62} = 452 \text{ psf}$$

From Figure E-3, with $\beta = 50°$ and $\frac{H_w}{H} = 0.33$, find $\mu_w = 0.93$.

Use layer thickness to average the unit weights. Unit weights are averaged only to the bottom of the critical circle.

$$\gamma_{avg} = \frac{\sum \gamma_i h_i}{\sum h_i} = \frac{(120)(10)+(100)(10)}{10+10} = 110$$

Calculate the driving force term P_d as follows:

$$P_d = \frac{\gamma H + q - \gamma_w H_w}{\mu_q \mu_w \mu_t} = \frac{(110)(24)+0-(62.4)(8)}{(1)(0.93)(1)} = 2302$$

From Figure E-1, with $d = 0$ and $\beta = 50°$, find $N_o = 5.8$:

Calculate the factor of safety using Equation E-7:

$$F = \frac{N_o c}{P_d} = \frac{(5.8)(452)}{2302} = 1.14$$

Example Problem E-2

Figure E-10 shows the same slope as in Figure E-9.

The deeper circle, tangent to elevation –20 ft, is analyzed as follows:

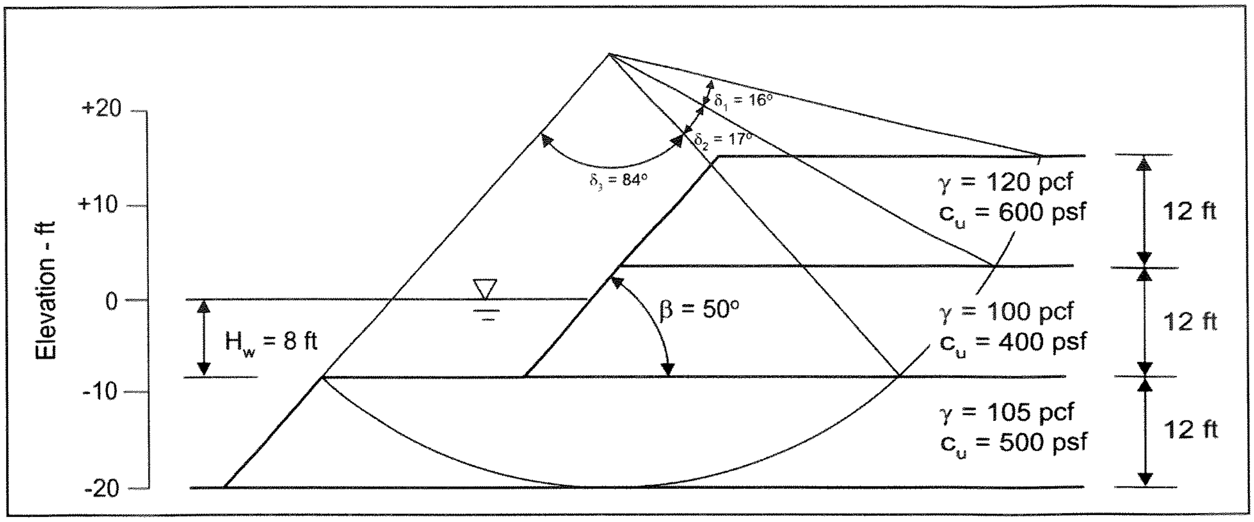


Figure E-10. Example for a circle tangent to elevation –20 ft for cohesive soil with φ = 0

For this circle:

$$d = \frac{D}{H} = \frac{12}{24} = 0.5$$

$$\frac{H_w}{H} = \frac{8}{24} = 0.33$$

Using the charts at the bottom of Figure E-1, with β = 50° and d = 0.5:

$x_o = 0.35$ and $y_o = 1.5$

$X_o = (H)(x_o) = (24)(0.35) = 8.4$ ft

$Y_o = (H)(y_o) = (24)(1.5) = 36$ ft

Plot the critical circle on the slope as shown in Figure E-10.

Measure the central angles of arc in each layer using a protractor. Calculate the weighted average strength parameter c_{avg}: using Equation E-1.

$$c_{avg} = \frac{\sum \delta_i c_i}{\sum \delta_i} = \frac{(16)(600) + (17)(400) + (84)(500)}{16 + 17 + 84} = 499 \text{ psf}$$

From Figure E-3, with d = 0.5 and $\frac{H_w}{H} = 0.33$:

$\mu_w = 0.95$

Use layer thickness to average the unit weights. Since the material below the toe of the slope is a $\phi = 0$ material, the unit weight is averaged only down to the toe of the slope. The unit weight below the toe has no influence on stability if $\phi = 0$.

$$\gamma_{avg} = \frac{\sum \gamma_i h_i}{\sum h_i} = \frac{(120)(10)+(100)(10)}{10+10} = 110$$

Calculate the driving force term P_d as follows:

$$P_d = \frac{\gamma H + q - \gamma_w H_w}{\mu_q \mu_w \mu_t} = \frac{(110)(24)+0-(62.4)(8)}{(1)(0.95)(1)} = 2253$$

From Figure E-1, with $d = 0.5$ and $\beta = 50°$, $N_o = 5.6$:

Calculate the factor of safety using Equation E-7:

$$F = \frac{N_o c}{P_d} = \frac{(5.6)(499)}{2253} = 1.24$$

This circle is less critical than the circle tangent to elevation –8 ft, analyzed previously.

Example Problem E-3

Figure E-11 shows a slope in soils with both c and ϕ. There are three layers with different strengths. There is no water outside the slope.

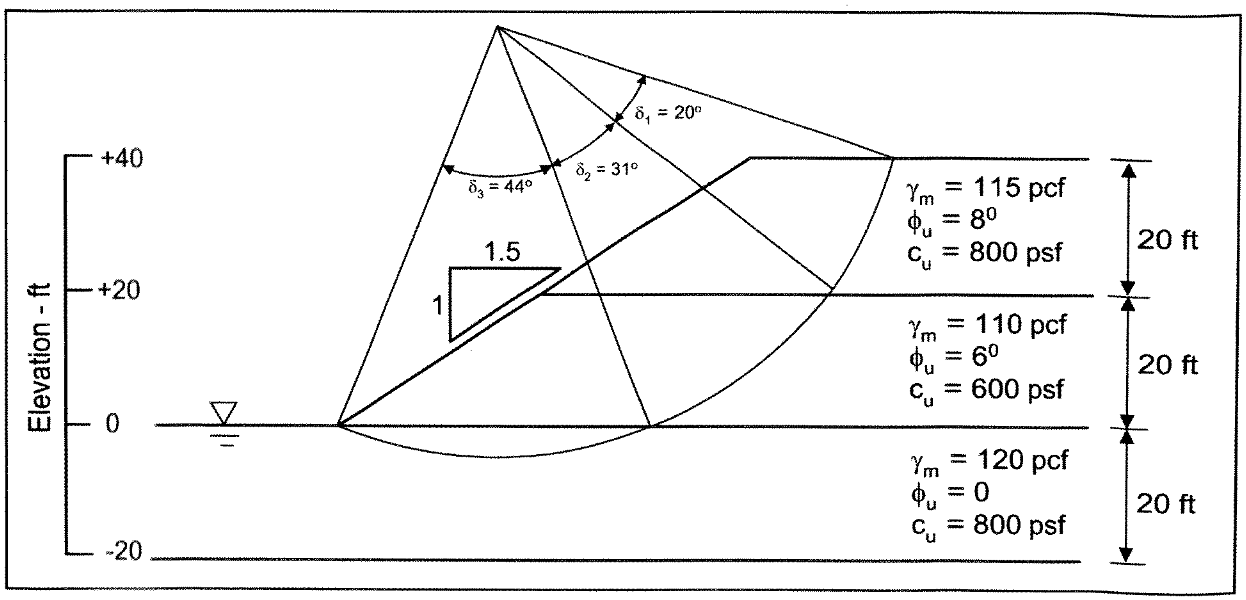


Figure E-11. Example for a total stress analysis of a toe circle in soils with both c and ϕ

The factor of safety for a toe circle is calculated as follows:

Use layer thickness to average the unit weights. Unit weights are averaged down to the toe of the slope, since the unit weight of the material below the toe has no effect on stability in this case.

$$\gamma_{avg} = \frac{\sum \gamma_i h_i}{\sum h_i} = \frac{(115)(10)+(110)(10)}{10+10} = 112.5$$

Since there is no surcharge, $\mu_q = 1$

Since there is no external water above toe, $\mu_w = 1$

Since there is no seepage, $\mu_w' = 1$

Since there are no tension cracks, $\mu_t = 1$

Calculate the driving force term P_d as follows:

$$P_d = \frac{\gamma H + q - \gamma_w H_w}{\mu_q \mu_w \mu_t} = \frac{(112.5)(40)}{(1)(1)(1)} = 4500 \text{ psf}$$

Calculate P_e as follows:

$$P_e = \frac{\gamma H + q - \gamma_w H_w'}{\mu_q \mu_w'} = \frac{(112.5)(40)}{(1)(1)} = 4500 \text{ psf}$$

Estimate $c_{avg} = 700$ psf and $\phi_{avg} = 7°$, and calculate $\lambda_{c\phi}$ as follows:

$$\lambda_{c\phi} = \frac{P_e \tan\phi}{c} = \frac{(4500)(0.122)}{700} = 0.8$$

From Figure E-5, with b = 1.5 and $\lambda_{c\phi} = 0.8$:

$x_o = 0.6$ and $y_o = 1.5$

$X_o = (H)(x_o) = (40)(0.6) = 24$ ft

$Y_o = (H)(y_o) = (40)(1.5) = 60$ ft

Plot the critical circle on the given slope, as shown in Figure E-11.

Calculate c_{avg}, $\tan\phi_{avg}$, and $\lambda_{c\phi}$ as follows:

$$c_{avg} = \frac{\sum \delta_i c_i}{\sum \delta_i} = \frac{(20)(800)+(31)(600)+(44)(800)}{20+31+44} = 735 \text{ psf}$$

$$\tan\phi_{avg} = \frac{\sum \delta_i \tan\phi_i}{\sum \delta_i} = \frac{(20)(\tan 8°) + (31)(\tan 6°) + (44)(\tan 0°)}{20 + 31 + 44} = 0.064$$

$$\lambda_{c\phi} = \frac{P_e \tan\phi}{c} = \frac{(4500)(0.064)}{735} = 0.4$$

From Figure E-5, with b = 1.5 and $\lambda_{c\phi} = 0.4$:

$x_o = 0.65$ and $y_o = 1.45$

$X_o = (H)(x_o) = (40)(0.65) = 26$ ft

$Y_o = (H)(y_o) = (40)(1.45) = 58$ ft

This circle is close to the previous iteration, so keep $\lambda_{c\phi} = 0.4$ and $c_{avg} = 735$ psf

From Figure E-5, with b = 1.5 and $\lambda_{c\phi} = 0.4$

$N_{cf} = 6$

Calculate the factor of safety as follows:

$$F = N_{cf}\frac{c}{P_d} = 6.0\frac{735}{4500} = 1.0$$

Example Problem E-4

Figure E-12 shows the same slope as shown in Figure E-11. Effective stress strength parameters are shown in the figure, and the analysis is performed using effective stresses. There is water outside the slope, and seepage within the slope.

Use layer thickness to average the unit weights. Unit weights are averaged only down to the toe of the slope.

$$\gamma_{avg} = \frac{\sum \gamma_i h_i}{\sum h_i} = \frac{(115)(10) + (115)(10)}{10 + 10} = 115$$

For this slope:

$$\frac{H_w}{H} = \frac{10}{40} = 0.25$$

$$\frac{H_w'}{H} = \frac{30}{40} = 0.75$$

Since there is no surcharge, $\mu_q = 1$

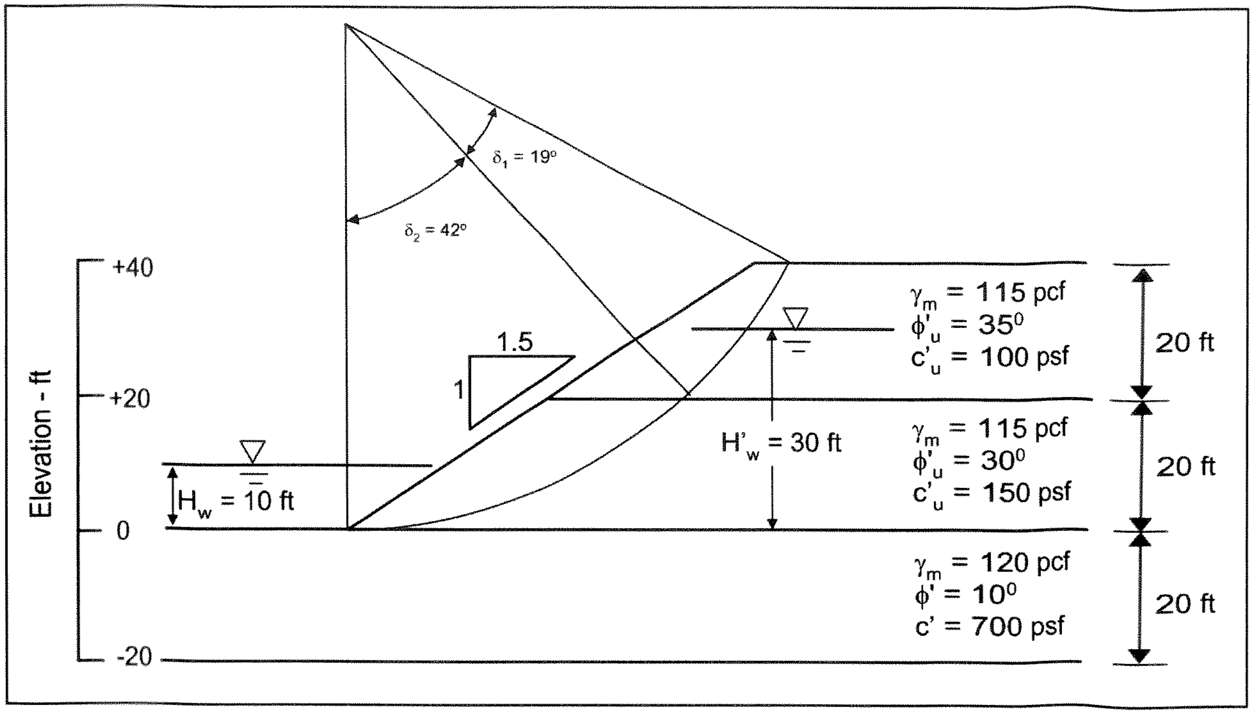


Figure E-12. Example for an effective stress analysis of a toe circle in soils with both c′ and φ′

Using Figure E-3 for toe circles, with $H_w/H = 0.25$ and $\beta = 33.7°$, find $\mu_w = 0.96$

Using Figure E-3 for toe circles, with $H_w'/H = 0.75$ and $\beta = 33.7°$, find $\mu_w' = 0.95$

Since there are no tension cracks, $\mu_t = 1$

Calculate the driving force term P_d as follows:

$$P_d = \frac{\gamma H + q - \gamma_w H_w}{\mu_q \mu_w \mu_t} = \frac{(115)(40) + 0 - (62.4)(10)}{(1)(0.96)(1)} = 4141 \text{ psf}$$

Calculate P_e as follows:

$$P_e = \frac{\gamma H + q - \gamma_w H_w'}{\mu_q \mu_w'} = \frac{(115)(40) + 0 - (62.4)(30)}{(1)(0.95)} = 2870 \text{ psf}$$

Estimate $c_{avg} = 120$ psf and $\phi_{avg} = 33°$

$$\lambda_{c\phi} = \frac{P_e \tan\phi}{c} = \frac{(2870)(0.64)}{120} = 15.3$$

From Figure E-5, with $b = 1.5$ and $\lambda_{c\phi} = 15.3$:

$x_o = 0$ and $y_o = 1.9$

$X_o = (H)(x_o) = (40)(0) = 0$ ft

$Y_o = (H)(y_o) = (40)(1.9) = 76$ ft

Plot the critical circle on the given slope as shown in Figure E-12.

Calculate c_{avg}, $\tan \phi_{avg}$, and $\lambda_{c\phi}$ as follows:

$$c_{avg} = \frac{\sum \delta_i c_i}{\sum \delta_i} = \frac{(19)(100) + (42)(150)}{19 + 42} = 134 \text{ psf}$$

$$\tan \phi_{avg} = \frac{\sum \delta_i \tan \phi_i}{\sum \delta_i} = \frac{(19)(\tan 35°) + (42)(\tan 30°)}{19 + 42} = 0.62$$

$$\lambda_{c\phi} = \frac{(2870)(0.62)}{134} = 13.3$$

From Figure E-5, with b = 1.5 and $\lambda_{c\phi} = 13.3$:

$x_o = 0.02$ and $y_o = 1.85$

$X_o = (H)(x_o) = (40)(0.02) = 0.8$ ft

$Y_o = (H)(y_o) = (40)(1.85) = 74$ ft

This circle is close to the previous iteration, so keep $\lambda_{c\phi} = 13.3$ and $c_{avg} = 134$ psf

From Figure E-5, with b = 1.5 and $\lambda_{c\phi} = 13.3$:

$N_{cf} = 35$

Calculate the factor of safety as follows:

$$F = N_{cf} \frac{c}{P_d} = 35 \frac{134}{4141} = 1.13$$

Example Problem E-5

Figure E-13 shows a slope where a relatively thin layer of soil overlies firm soil. The critical failure mechanism for this example is sliding along a plane parallel to the slope, at the top of the firm layer. This slope can be analyzed using the infinite slope stability chart shown in Figure E-7.

Calculate the factor of safety for seepage parallel to the slope and for horizontal seepage emerging from the slope.

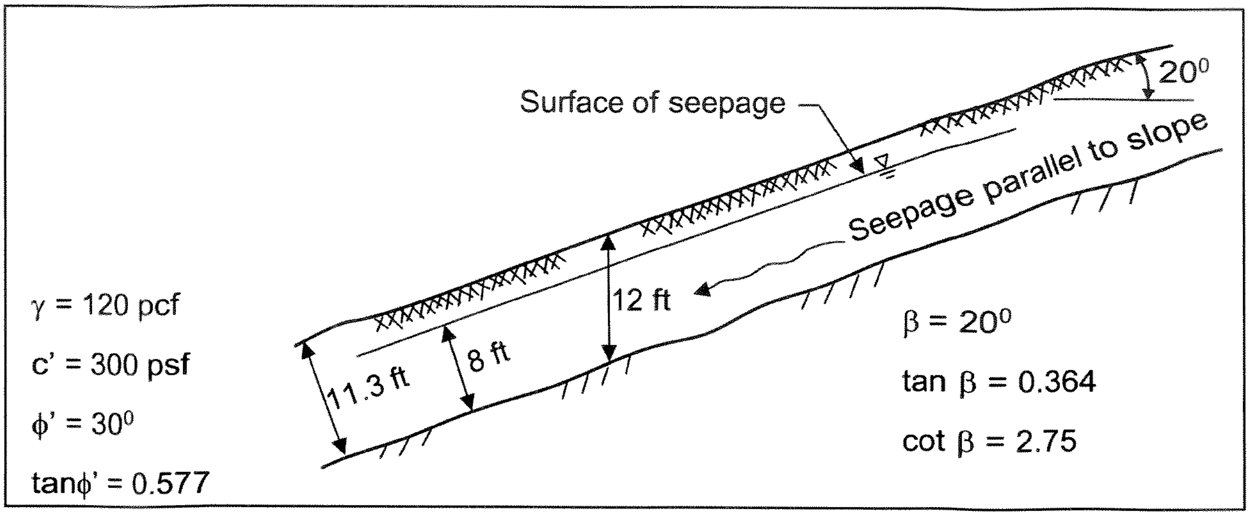


Figure E-13. Example of an infinite slope analysis

For seepage parallel to slope:

X = 8 ft and T = 11.3 ft

$$r_u = \frac{X}{T}\frac{\gamma_w}{\gamma}\cos^2\beta = \frac{8}{11.3}\frac{62.4}{120}(0.94)^2 = 0.325$$

From Figure E-7, with r_u = 0.325 and cot β = 2.75:

A = 0.62 and B = 3.1

Calculate the factor of safety, as follows:

$$F = A\frac{\tan\phi'}{\tan\beta} + B\frac{c'}{\gamma H} = 0.62\frac{0.577}{0.364} + 3.1\frac{300}{(120)(12)} = 0.98 + 0.65 = 1.63$$

For horizontal seepage emerging from slope, $\theta = 0°$

$$r_u = \frac{\gamma_w}{\gamma}\frac{1}{1+\tan\beta\tan\theta} = \frac{62.4}{120}\frac{1}{1+(0.364)(0)} = 0.52$$

From Figure E-7, with r_u = 0.52 and cot β = 2.75:

A = 0.41 and B = 3.1

Calculate the factor of safety, as follows:

$$F = A\frac{\tan\phi'}{\tan\beta} + B\frac{c'}{\gamma H} = 0.41\frac{0.577}{0.364} + 3.1\frac{300}{(120)(12)} = 0.65 + 0.65 = 1.30$$

Note that the factor of safety for seepage emerging from the slope is smaller than the factor of safety for seepage parallel to the slope.

Example Problem E-6

Figure E-14 shows a submerged clay slope with $\phi = 0$ and strength is increasing linearly with depth.

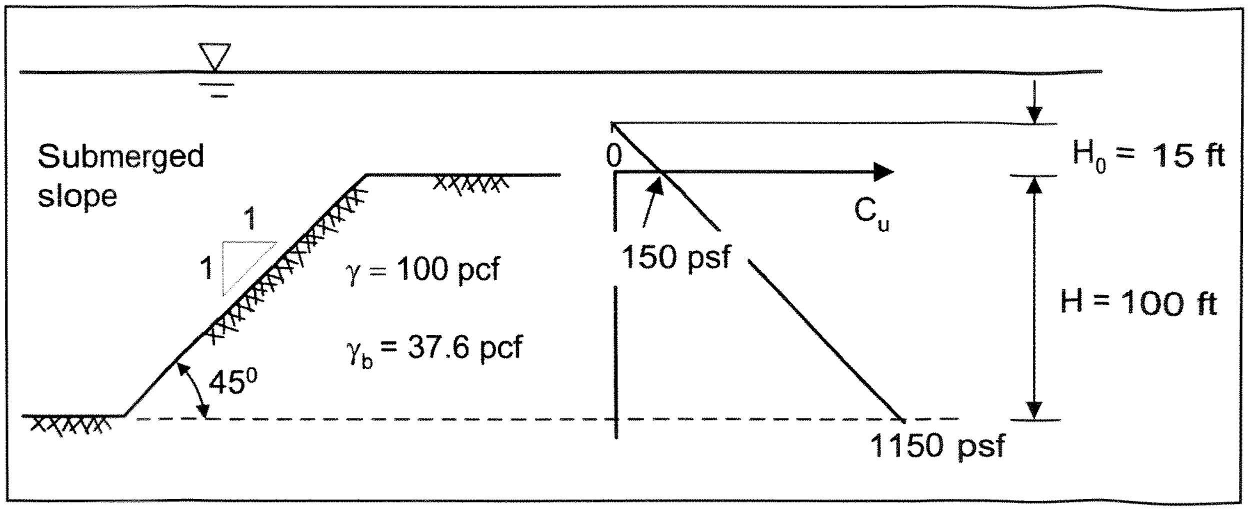


Figure E-14. Example of ϕ = 0, and strength increasing with depth

The factor of safety is calculated using the slope stability chart shown in Figure E-8.

Extrapolating the strength profile up to zero gives $H_o = 15$ ft

Calculate M as follows:

$$M = \frac{H_o}{H} = \frac{15}{100} = 0.15$$

From Figure E-8, with $M = 0.15$ and $\beta = 45^\circ$:

$N = 5.1$

From the soil strength profile, $c_b = 1150$ psf:

Calculate the factor of safety as follows:

$$F = N\frac{c_b}{\gamma(H + H_o)} = (5.1)\frac{1150}{(37.6)(115)} = 1.36$$

Appendix A
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Appendix B
Notation

Dimensions: F indicates force, L indicates length

A	=	cross-sectional area of slice (L^2)
b	=	width of slice (L)
b	=	slope ratio = cot β (dimensionless)
c	=	cohesion intercept for Mohr-Coulomb diagram plotted in terms of total normal stress, σ (F/L^2)
c'	=	cohesion intercept for Mohr-Coulomb diagram plotted in terms of total effective stress, σ' (F/L^2)
c_R	=	cohesion intercept as determined from the R envelope (F/L^2)
c_b	=	cohesion at the base of an embankment (F/L^2)
c_{avg}	=	average cohesion over length of slip surface (F/L^2)
c_D	=	'developed' or 'mobilized' cohesion (F/L^2)
c'_D	=	'developed' or 'mobilized' cohesion (F/L^2)
C_D	=	force because of the 'developed' or 'mobilized' cohesion on base of slice (F)
C_1	=	term used to calculate side forces on a slice (F)
C_2	=	term used to calculate side forces on a slice (F)
C_3	=	term used to calculate side forces on a slice (F)
C_4	=	term used to calculate side forces on a slice (F)
d	=	depth factor = D/H (dimensionless)
d	=	intercept value of failure envelope on 'p-q' diagram (F/L^2)
d_h	=	horizontal moment arm (L)
d_V	=	vertical moment arm (L)
d'	=	intercept value of failure envelope on (σ_1-σ_3) vs. σ_3 'modified' Mohr-Coulomb diagram (F/L^2)
d_R	=	intercept value of R failure envelope on 'p-q' diagram (F/L^2)
d_{crack}	=	depth of vertical 'tension' crack (L)
$d_{Kc=1}$	=	intercept value for τ_{ff} vs. σ'_{fc} shear strength envelope for isotropic consolidation (F/L^2)
D	=	depth from toe of slope to lowest point on the slip circle (L)
e	=	void ratio (dimensionless)
E	=	horizontal component of the interslice force (F)
E_A	=	active force acting on a wedge (F)
E_P	=	passive force acting on a wedge (F)
F	=	factor of safety (defined with respect to shear strength) (dimensionless)

FS	=	factor of safety (defined with respect to shear strength) (dimensionless)
F_D	=	resultant force because of total normal force, N, and developed frictional resistance, N tan ϕ_D, on the bottom of a slice (F)
h_{avg}	=	average height of slice (L)
h_i	=	piezometric height above the bottom of the slice (pressure head) at the upslope side of the slice (L)
h_{i+1}	=	piezometric height above the bottom of the slice (pressure head) at the downslope side of the slice (L)
h_p	=	piezometric height above the bottom of the slice (pressure head) at the midpoint of the slice (L)
h_s	=	average height of water above top of slice (L)
H	=	height of the slope (L)
H_0	=	height above slope where soil strength vs. depth intersects zero (L)
H_w	=	depth of water outside slope (L)
H'_w	=	height of water within slope (L)
K_o	=	at-rest pressure coefficient (dimensionless)
K_A	=	active earth pressure coefficient (dimensionless)
K_P	=	passive earth pressure coefficient (dimensionless)
K_c	=	effective principal stress ratio, $\sigma'_{1c}/ \sigma'_{3c}$, for consolidation (dimensionless)
K_f	=	effective principal stress ratio, $\sigma'_{1f}/\sigma'_{3f}$ at failure (dimensionless)
ℓ_{top}	=	length of the top of the slice (L)
m_α	=	term used in the Simplified Bishop method (dimensionless)
M	=	term to relate height above slope where soil strength is zero to height of slope
M_P	=	moment produced by the force P about the center of the circle (FL)
n_α	=	term used to calculate side forces on a slice (dimensionless)
N	=	total normal force on the bottom of the slice (F)
N	=	stability number for an embankment with an uniform increase in cohesion with depth (dimensionless)
N'	=	effective normal force on the bottom of the slice (F)
N_0	=	stability number for a homogeneous embankment and foundation overlying a rigid boundary with ϕ_u=0 (dimensionless)
N_{cf}	=	stability number for a homogeneous embankment and foundation overlying a rigid boundary with ϕ>0 (dimensionless)
p	=	total stress state variable used to plot 'modified' Mohr-Coulomb diagrams, usually $\frac{1}{2}(\sigma_1+\sigma_3)$ but sometimes used to represent $\frac{1}{3}(\sigma_1+\sigma_2+\sigma_3)$ (F/L^2)
p'	=	effective stress state variable used to plot 'modified' Mohr-Coulomb diagrams, usually $\frac{1}{2}(\sigma_1+\sigma_3)$ but sometimes used to represent $\frac{1}{3}(\sigma_1+\sigma_2+\sigma_3)$ (F/L^2)
p_{avg}	=	average water pressure on top of slice (F/L^2)

P = water force on top of slice, acting perpendicular to top of slice (F)

P_d = term to account for weight of slope and surcharge loading, submergence, and tension crack corrections (F/L^2)

P_e = term to account for the effects of water within the slope (F/L^2)

q = surcharge load (F/L^2)

q = stress state variable used to plot 'modified' Mohr-Coulomb diagrams, usually $\frac{1}{2}(\sigma_1-\sigma_3)$ but sometimes used to represent $(\sigma_1-\sigma_3)$ (F/L^2)

q_{ult} = ultimate bearing capacity (F/L^2)

R = radius of the circle (L)

R = resultant force because of weight of slice and water pressures on top, sides and bottom of slice (F)

s = shear strength (F/L^2)

s_d = drained shear strength (F/L^2)

$s_{passive}$ = shear strength based on active Rankine stress state (F/L^2)

s_2 = shear strength used for second stage of computations for rapid drawdown (F/L^2)

S = shear force on the bottom of the slice (F)

u = pore water pressure (F/L^2)

u_{bp} = back pressure – pore water pressure for consolidation and drained shear in the triaxial test (F/L^2)

u_c = pore water pressure at consolidation (F/L^2)

u_f = pore water pressure at failure (F/L^2)

U_b = force resulting from pore water pressure on the bottom of the slice (F)

U_i = force resulting from water pressures on the upslope side of the slice (F)

U_{i+1} = force resulting from water pressures on the downslope side of the slice (F)

U_L = water force on left of slice (F)

U_R = water force on right of slice (F)

W = weight of slice (F)

W' = effective weight of slice (F)

X = shear component of the interslice force (F)

y_t = location of side forces (L)

z = vertical depth to a plane parallel to slope (L)

Z_i = interslice force on the upslope side of the slice (F)

Z_{i+1} = interslice force on the downslope side of the slice (F)

z_t = depth of tensile stresses (L)

α = inclination from horizontal of the bottom of the slice (degrees)

α_s = angle between flow lines and embankment face (degrees)

β = inclination from horizontal of the top o [illegible] degrees)

δ = inclination of the earth pressure force (degrees)

$\Delta\ell$ = length of bottom of slice (L)

Δu = change in pore water pressure, usually during shear (F/L^2)

Δx = width of slice (F)

$\Delta\phi$ = change in friction angle (degrees)

γ = total unit weight of soil (F/L^3)

γ' = submerged unit weight of soil (F/L^3)

γ_m = moist unit weight of soil (F/L^3)

γ_w = unit weight of water (F/L^3)

γ_{sat} = saturated unit weight of soil (F/L^3)

$\lambda_{c\phi}$ = term to relate P_c with shear strength parameters (dimensionless)

μ_q = surcharge correction factor (dimensionless)

μ_w = submergence correction factor (dimensionless)

μ_t = tension crack correction factor (dimensionless)

μ'_w = seepage correction factor (dimensionless)

Ω = term used to calculate the inclination of the critical slip surface (dimensionless)

ϕ = angle of internal friction for Mohr-Coulomb diagram plotted in terms of total normal stress, σ (degrees)

ϕ' = angle of internal friction for Mohr-Coulomb diagram plotted in terms of effective normal stress, σ' (degrees)

ϕ_D = ‘developed’ or ‘mobilized’ total stress angle of internal friction (degrees)

ϕ'_D = ‘developed’ or ‘mobilized’ effective stress angle of internal friction (degrees)

ϕ_R = angle of internal friction as determined from the R envelope (degrees)

ϕ_u = undrained friction angle (degrees)

ϕ_{secant} = secant value of friction angle ($\tan \phi_{secant} = \tau_f/\sigma_f$) (degrees)

σ = total normal stress (F/L^2)

σ' = effective normal stress (F/L^2)

σ'_v = effective vertical stress (F/L^2)

σ'_{vc} = effective vertical stress for consolidation (F/L^2)

σ_{1f} = major principal total stress at failure (F/L^2)

σ'_{1f} = major principal effective stress at failure (F/L^2)

σ'_{1c} = effective major principal stress for consolidation (F/L^2)

σ_{3f} = minor principal total stress at failure (F/L^2)

σ'_{3f} = minor principal effective stress at failure (F/L^2)

σ'_{3c} = minor principal effective stress for consolidation (F/L^2)

$\sigma'_{3\text{-critical}}$ = 'critical' effective confining pressure for critical state (F/L^2)

σ'_c = effective normal stress on the slip surface at consolidation, before rapid drawdown (F/L^2)

σ'_d = effective normal stress on the slip surface after drainage following rapid drawdown (F/L^2)

σ'_{fc} = effective normal stress on the failure plane at consolidation (F/L^2)

σ_i = normal stress where R and S envelopes intersect (F/L^2)

σ'_1/σ'_3 = principal stress ratio (dimensionless)

$(\sigma_1\text{-}\sigma_3)$ = principal stress difference (F/L^2)

$(\sigma_1\text{-}\sigma_3)_f$ = principal stress difference at failure (F/L^2)

τ = shear stress (F/L^2)

τ_c = shear stress on the slip surface at consolidation, before rapid drawdown (F/L^2)

τ_{fc} = shear stress on the failure plane at consolidation (F/L^2)

τ_{ff} = shear stress on the failure plane at failure (F/L^2)

$\tau_{ff\text{-}Kc=1}$ = shear stress (strength) from envelope of τ_{ff} vs. σ'_{fc} for isotropic consolidation (F/L^2)

$\tau_{ff\text{-}Kc=Kf}$ = shear stress (strength) from envelope of τ_{ff} vs. σ'_{fc} for maximum degree of anisotropic consolidation, $K_c = K_f$ (F/L^2)

θ = inclination of the interslice force (degrees)

Ψ = angle of inclination of failure envelope on 'p-q' diagram (degrees)

Ψ' = angle of inclination of failure envelope on $(\sigma_1\text{-}\sigma_3)$ vs. σ_3 'modified' Mohr-Coulomb diagram (degrees)

Ψ_R = angle of inclination of the R failure envelope on 'p-q' diagram (degrees)

$\Psi_{Kc=1}$ = slope angle for τ_{ff} vs. σ'_{fc} shear strength envelope for isotropic consolidation (degrees)

Appendix C

MSE WALLS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

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APPENDIX C

MECHANICALLY STABILIZED EARTH WALL

DESIGN GUIDELINES

C.1 INTRODUCTION

This document outlines SCDOT's design methodology for Mechanically Stabilized Earth (MSE) Walls. MSE wall structures are internally stabilized fill walls constructed of alternating layers of compacted soil and reinforcement. The design of MSE walls follows the design steps provided in Chapter 18. This Appendix governs the design of permanent and temporary MSE wall structures. The design life of both permanent and temporary MSE walls is provided in Chapter 18. The design responsibilities of the SCDOT (or its representative) and the MSE wall supplier are outlined with respect to external and internal stability of the MSE wall structure.

C.2 DESIGN CONSIDERATIONS AND REQUIREMENTS

The first part of the design is determining if an MSE wall is appropriate for the application being planned. If an MSE wall is appropriate, determine the geometry, the external loading conditions, the performance criteria and any construction constraints. The geometry should include the location relative to the remainder of the project (i.e. to the centerline and specific station) and should establish wall stationing as needed. The geometry should also indicate the anticipated top and base of the wall, as well as slopes that tie into the wall. During this step of the design process, external loads should be identified. These loads include, but are not limited to transient (traffic), permanent (weight of pavement surface) and/or seismically induced loads. The performance criteria are based on the Operational Classification of the Bridge or Roadway (see Chapter 8). The performance limits are provided in Chapter 10. Any constraints on construction should also be identified during this step (for example, soft ground, standing water, limited ROW, utilities, etc). These construction constraints should be carefully considered before deciding to use an MSE wall.

C.3 SITE CONDITIONS

The second step in the design of MSE walls is the evaluation of the topography, subsurface conditions, in-situ soil/rock parameters and the parameters for the backfill. The evaluation of the topography should include reviewing the height requirements of the wall, the amount of space between the front of the MSE wall and the anticipated extent of the reinforcement and the condition of the existing ground surface. This evaluation should identify the need for any temporary shoring that may be required to install the MSE wall (i.e. the grading of the site requires cutting). The subsurface conditions and in-situ soil/rock parameters shall be evaluated using the procedures presented in Chapters 4 through 7. The reinforced backfill to be used to construct the MSE wall shall meet the criteria provided below.

Table C-1, MSE Wall Reinforced Backfill Properties

Material Property	Granular Backfill	Stone Backfill
Internal Friction Angle[1]	32° - 34°	36° - 38°
Total Unit Weight (lbs./cubic foot)	120	110

[1]Based on Triaxial testing of samples recompacted to 95 percent of the Standard Proctor

Table C-2, Granular Backfill Gradation Requirements

Sieve Size	Percent Passing
1-1/2 in[1]	100
¾ in[2]	
No. 40	0-60
No. 100	0-30
No. 200	0-15
Plasticity Index	≤ 6
Liquid Limit	≤ 30
C_U[3]	≥ 4[4]
Organic Content	< 1%

[1]Inextensible reinforcement
[2]Extensible reinforcement
[3]$C_U = D_{60}/D_{10}$
[4]Pullout or additional internal friction testing required for C_U less than 4

Table C-3, Stone Backfill Gradation Requirements

Reinforcement Material	Coarse Aggregate No.[1]
Geosynthetic [2]	67, 6M
Metallic[3]	5, 57, 67, 6M

[1]Meets the requirements of the SCDOT Standard Construction Specifications (latest edition)
[2]Extensible reinforcement (polyester and/or polyolefin)
[3]Inextensible reinforcement

Table C-4, Electrochemical Properties of Reinforced Backfill

Reinforcement Material	Property	Criteria
Metallic	Resistivity[1]	>3,000 ohm-cm
Metallic	Chlorides	<100 ppm
Metallic	Sulfates	<200 ppm
Metallic/Geosynthetic[2]	pH	3.5 < pH < 9
Metallic/Geosynthetic[3]	pH	4.5 < pH < 10

[1]Chloride and Sulfate testing are not required if the resistivity is greater than 5,000 ohm-cm
[2]Granular Backfill
[3]Stone Backfill

Table C-5, Temporary MSE Granular Backfill Properties

Gradation		Plasticity Index	Liquid Limit	C_U	Internal Friction Angle	Total Unit Weight (pcf)
¾ in	No. 200	≤ 15	≤ 30	≥ 4	28° - 30°	115
100%	$\leq 30\%$					

C.4 INITIAL WALL GEOMETRY

The third step in the design of MSE walls is establishing the initial geometry of the MSE wall. Figure C-1 provides the terminology for MSE wall geometry. The height (H) of an MSE wall is measured vertically from the top of the MSE wall to the top of the leveling pad. MSE wall structures, with panel type facings, should not exceed heights of 40 feet, and with modular block type facings, should not exceed heights of 30 feet. Wall heights in excess of these limits will require approval from the GDS and the PCS/GDS. The length of reinforcement (L) is measured from the back of MSE wall panels. Alternately, the length of reinforcement (B) is measured from the front face for modular block type MSE walls. The minimum reinforcement length is 0.7H or 8 feet whichever is greater. MSE wall structures with sloping surcharge fills or other concentrated loads will generally require longer reinforcement lengths of 0.8H to 1.1H. MSE walls may be built to heights mentioned above; however, the external stability requirements may limit MSE wall height due to bearing capacity, settlement, or stability problems.

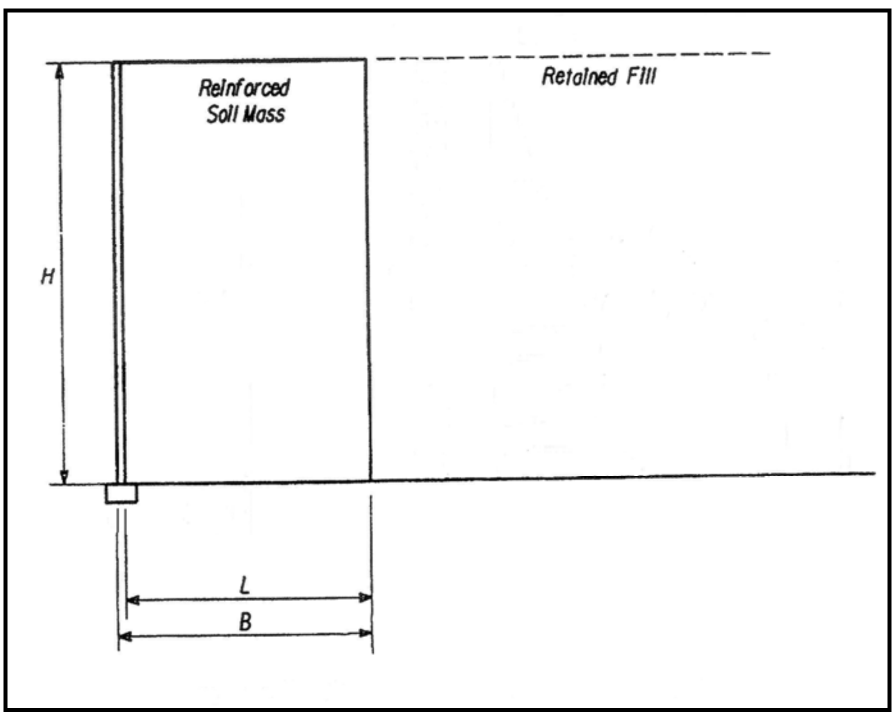


Figure C-1, MSE Wall Schematic (Mechanically Stabilized Earth walls and Reinforced Soil Slopes Design & Construction Guidelines – March 2001)

The top of the leveling pad will require a minimum embedment below finished grade of 2 feet. Greater embedment depths may be required due to bearing capacity, settlement, stability, erosion, or scour requirements and if utilities, ditches, or other structures are located adjacent to the wall. The minimum embedment depths based on local bearing capacity considerations taking into account the geometry in front of the wall are presented in Table C-6.

Table C-6, Minimum MSE Wall Embedment Depth Based on Local Bearing Capacity

Slope in Front of Wall	Minimum Embedment Depth
Horizontal (walls)	H/20
Horizontal (abutments)	H/10
3H:1V	H/10
2H:1V	H/7
1.5H:1V	H/5

A minimum horizontal bench of 4 feet is required in front of the MSE wall structure, for MSE walls built on slopes. This minimum bench is required to protect against local instability at the base of the wall.

C.5 EXTERNAL STABILITY

The external stability analysis develops the unfactored and factored loads on an MSE wall and checks the eccentricity, sliding and bearing resistances. The external stability analyses cover the fourth step to the eighth step of the design process provided in Chapter 18. These analyses are normally performed by the geotechnical engineer-of-record, whether SCDOT personnel or consultant.

C.5.1 Unfactored Load Estimate

In this step, the geotechnical engineer-of-record is responsible for developing the unfactored loads that are used in the design of the MSE wall. These loads are the result of earth pressures induced by the retained fill materials and any surcharge loadings. The earth pressure loadings include the horizontal and vertical earth pressures and any soil surcharge loadings. There are three cases for the development of earth pressures; these are 1) horizontal backslope with traffic surcharge; 2) sloping backslope; and, 3) broken backslope. The surcharge loadings can include vehicle live loads, the loads imposed by a bridge, etc. These loading conditions are discussed in Chapter 8. In addition, Chapter 8 also provides some unit weights for materials that are used as surcharges. If a bridge is to be supported by shallow foundations, no portion of the spread footings shall be located above or within the reinforced soil mass of MSE walls.

C.5.1.1 Horizontal Backslope with Traffic Surcharge

The procedure for estimating the earth pressures acting on the back of the reinforced soil mass for the horizontal backslope with traffic surcharge is depicted in Figure C-2. The active earth pressure coefficient (K_a) for vertical walls (i.e., walls with less than 8° batter) with horizontal backfill is calculated according to the procedures provided in Chapter 18. When considering live load on MSE walls for this condition, the factored surcharge load is generally included over the reinforced soil mass during the evaluation of foundation bearing resistance, overall (global) stability and tensile resistance of the reinforcement (see Figure C-2). The live load surcharge is not included over the reinforced soil mass in the evaluation of eccentricity, sliding, reinforcement pullout or other failure mechanisms for which the surcharge load increases the resistance to failure (i.e. increases stability). If the surcharge consists of point loads, Earth Retaining Structures, FHWA NHI-07-071, June, 2008.

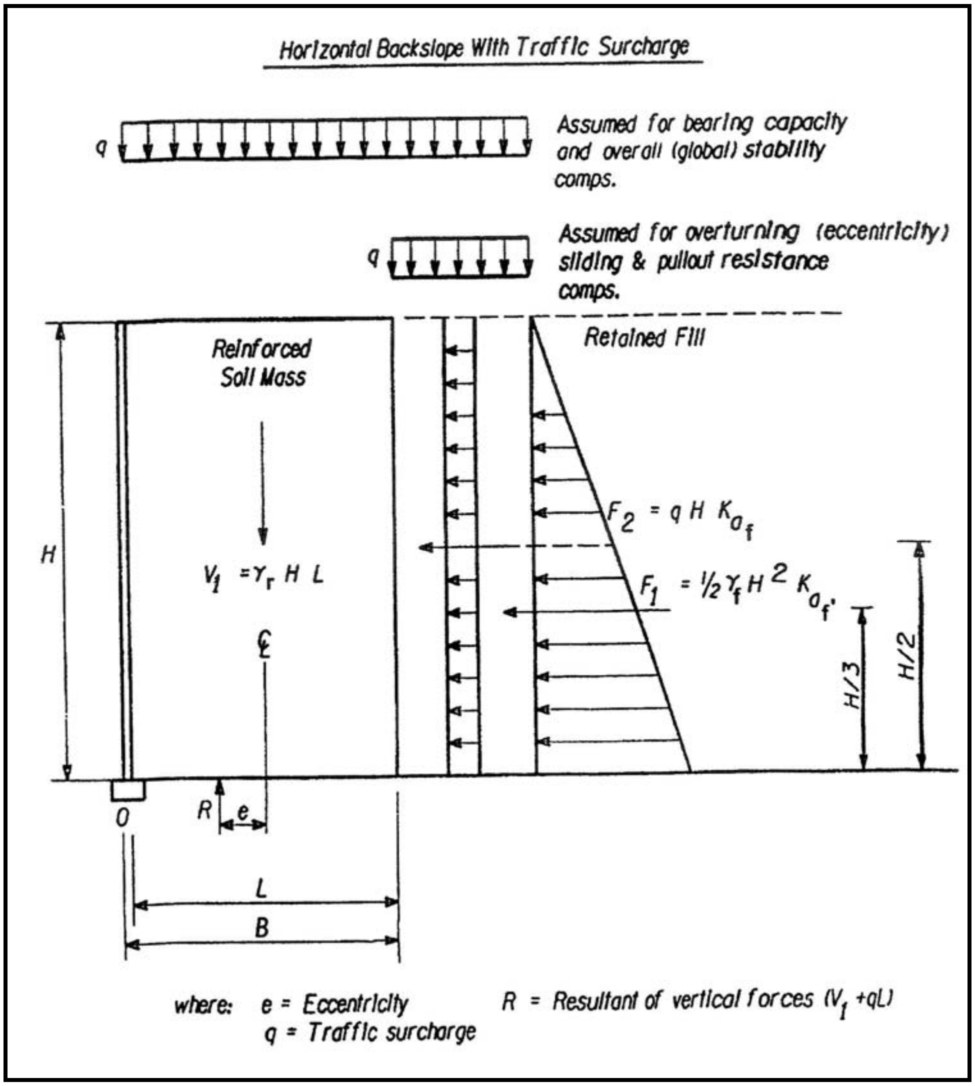


Figure C-2, MSE Wall Earth Pressure for Horizontal Backslope With Traffic Surcharge (Mechanically Stabilized Earth walls and Reinforced Soil Slopes Design & Construction Guidelines – March 2001)

C.5.1.2 Sloping Backslope

The active earth pressure coefficient (K_a) changes when there is a slope behind the MSE wall. K_a is determined using the following equation:

$$K_a = \frac{\sin^2(\theta + \varphi')}{\Gamma \sin^2 \theta \sin(\theta - \delta)}$$ Equation C-1

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\varphi' + \delta)\sin(\varphi' - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}} \right]^2$$ Equation C-2

Where,

β = Nominal slope angle of backfill behind wall (see Figure C-3)

δ = Angle of wall friction

φ' = Friction angle of retained fill

θ = 90° for vertical wall

The force on the rear of the reinforced soil mass (F_T) and the resulting horizontal (F_H) and vertical (F_V) forces are determined from the following equations:

$$F_T = \frac{1}{2}\gamma h^2 K_a$$ Equation C-3

$$F_H = F_T \cos\beta$$ Equation C-4

$$F_V = F_T \sin\beta$$ Equation C-5

Where,

γ = Unit weight of retained fill material

h = See Figure C-3

K_a = Determined in Equation C-1

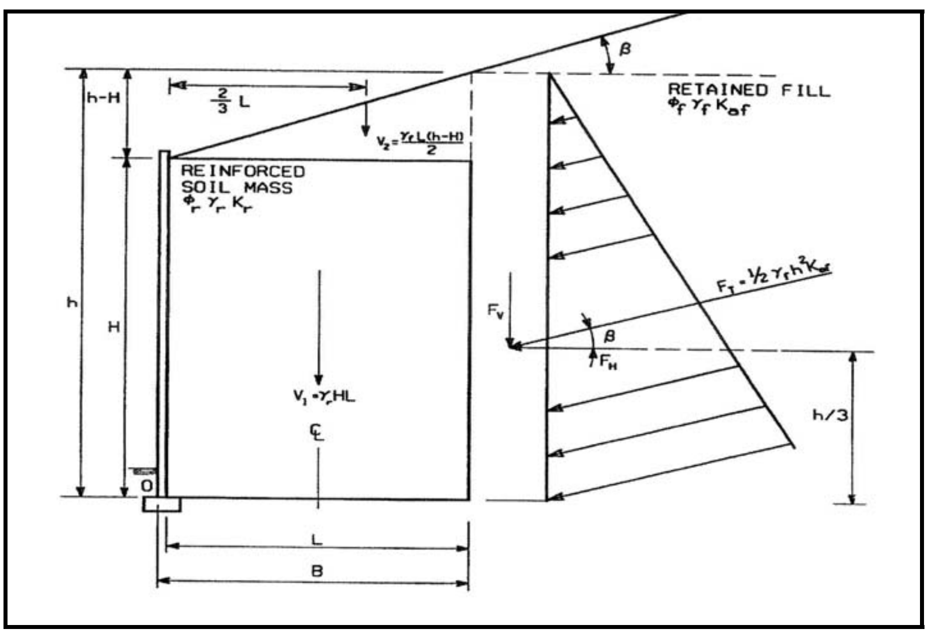


Figure C-3, MSE Wall Earth Pressure for Sloping Backfill (Mechanically Stabilized Earth walls and Reinforced Soil Slopes Design & Construction Guidelines – March 2001)

C.5.1.3 Broken Backslope

For broken backslopes (see Figure C-4), the active earth pressure coefficient (K_a) is determined using Equation C-1. The force acting on the rear of the MSE wall, F_T, is determined using Equation C-3.

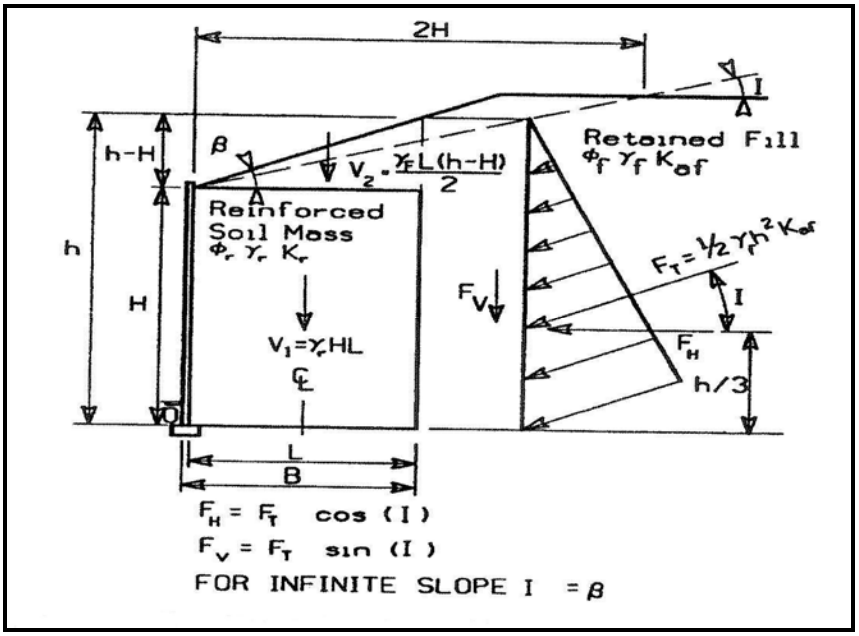


Figure C-4, MSE Wall Earth Pressure for Broken Backslope (Mechanically Stabilized Earth walls and Reinforced Soil Slopes Design & Construction Guidelines – March 2001)

C.5.2 Factored Loads

Portions of the following sections of this Appendix are adopted directly from Earth Retaining Structures – June 2008 and are used with the permission of the US Department of Transportation, Federal Highway Administration. The italics are added to reflect additions or modifications to the selected text and to supply references to this Manual. According to Earth Retaining Structures:

> …the unfactored loads from *the previous step* are multiplied by load factors to obtain the factored loads for each limit state. The load factors for the limit state are provided in *Chapter 8*.
>
> Load factors for permanent loads are selected to produce the maximum destabilizing effect for the design check being considered. For example, to produce the maximum destabilizing effect, when checking sliding resistance, γ_{EV} is selected as the minimum value from *Table 8-6* (i.e., γ_{EV} = 1.00) and when checking bearing resistance, γ_{EV} is selected as the maximum value from *Table 8-6* (i.e., γ_{EV} = 1.35)."

C.5.3 Eccentricity

According to Earth Retaining Structures:

The eccentricity of the wall (e_B) can be calculated for each load group as:

$$e_B = \frac{B}{2} - X_o \qquad \text{Equation C-6}$$

Where,

B = Base width (length of reinforcement elements)
X_o = Location of the resultant from the toe of the wall (*see Equation C-7*)

The parameter X_o is calculated as:

$$X_o = \frac{(M_{EV} - M_{HTOT})}{P_{EV}} \qquad \text{Equation C-7}$$

Where,

M_{EV} = Resisting moment due to factored vertical earth pressure calculated about the toe of the wall
M_{HTOT} = Driving moment due to factored horizontal earth pressure from ground and factored live load surcharge calculated about the toe of the wall
P_{EV} = Factored resultant force from vertical earth pressure due to the weight of reinforced soil

M_{EV} and M_{HTOT} are calculated using the factored loads. Example calculations for two different backslopes are given below:

For horizontal backslope with traffic surcharge (*Figure C-5*) condition:

$$M_{EV} = P_{EV}\frac{L}{2} \qquad \text{Equation C-8}$$

$$M_{HTOT} = \left(P_{EH}\frac{H}{3}\right) + \left(P_{LSH}\frac{H}{2}\right) \qquad \text{Equation C-9}$$

Where,

P_{EH} = Factored resultant force from horizontal earth pressure
P_{LSH} = Factored resultant force from horizontal component of surcharge load

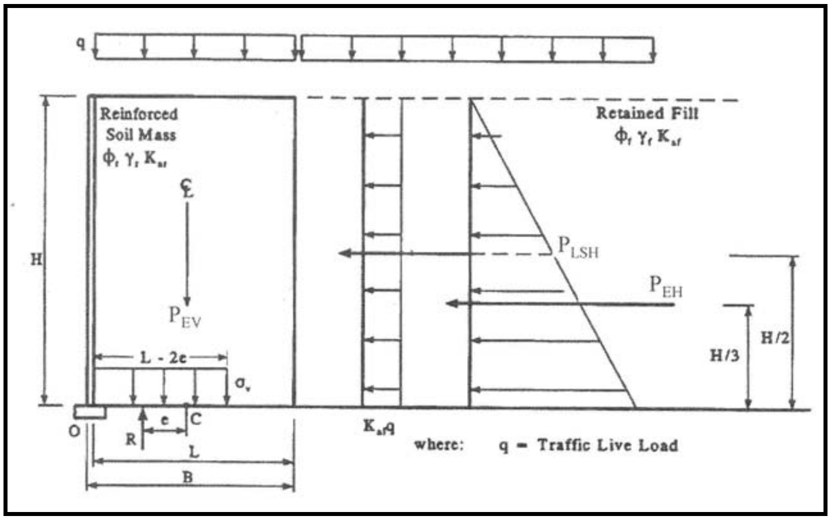


Figure C-5, MSE Wall Eccentricity for Horizontal Backslope With Traffic Surcharge (Earth Retaining Structures – June 2008)

It should be noted that the effect of external loadings on the MSE mass which increases sliding resistance, should only be included if the loadings are permanent. For example, live load traffic surcharges should be excluded.

For sloping backslope (*Figure C-6*) condition:

$$M_{EV} = P_{EV1}\frac{L}{2} + P_{EV2}\frac{2L}{3} + P_{EH}\sin\beta L \qquad \text{Equation C-10}$$

$$M_{HTOT} = P_{EH}\cos\beta\frac{h}{3} \qquad \text{Equation C-11}$$

Where,

P_{EV2} = Factored resultant force from earth pressure due to the weight of soil of sloping backslope

$P_{EH}\sin\beta$ = Factored resultant force from vertical component of the earth pressure

$P_{EH}\cos\beta$ = Factored resultant force from horizontal component of the earth pressure

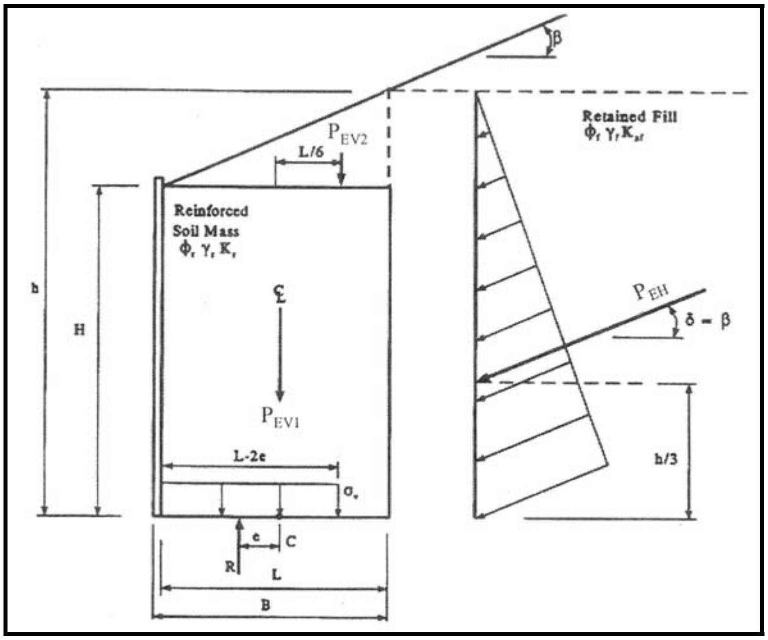


Figure C-6, MSE Wall Eccentricity for Sloping Backfill (Earth Retaining Structures – June 2008)

For Eccentricity to be considered acceptable, the calculated location of the resultant vertical force (based on factored loads) should be within the middle one-half of the base width for soil foundations (i.e., e_{max} = B/4) and middle three-fourths of the base width for rock foundations (i.e., e_{max} = 3B/8). Therefore, for each load group, e_B must be less than e_{max}. If e_B is greater than e_{max}, a longer length of reinforcement is required.

C.5.4 Sliding Resistance

Step 7 of the MSE wall design consists of checking for sliding resistance. According to Earth Retaining Structures:

Sliding and overall stability usually govern the design of structures greater than about 30 feet high, structures constructed on weak foundation soils, or structures with a sloping surcharge.

The live load surcharge is not considered as a stabilizing force when checking sliding. The driving forces in a sliding evaluation will generally include factored horizontal loads due to earth, water, seismic, and surcharge pressures and the resisting force is provided by the minimum shear resistance between the base of the MSE wall and foundation soil.

Sliding along the base of the wall is evaluated using the procedures in *Chapter 15* for spread footings.

It should be noted that any passive resistance provided by soil at the toe of the wall by embedment is ignored due to the potential for the soil to be removed through natural or manmade processes during the service life of the structure. The shear strength of the facing system is also conservatively neglected in most cases.

If the soil beneath the wall is cohesionless, the nominal sliding resistance (R_τ) between the soil and foundation is:

$$R_\tau = P_{EV} \tan\delta \qquad \text{Equation C-12}$$

Where,

P_{EV} = Minimum factored vertical load for the Strength limit state being considered
δ = Coefficient of sliding friction at the base of the reinforced soil mass

For continuous sheet reinforcement, δ is selected as the minimum of:

(1) Friction angle of reinforced fill;
(2) Friction angle of foundation soil; or
(3) Interface friction angle between the reinforcement and soil.

For discontinuous reinforcement (e.g., geogrid), δ is selected as the minimum value of (1) or (2) *above*.

Sliding resistance (R_τ) of the MSE wall is considered adequate if R_τ is equal to or greater than the maximum factored horizontal earth pressure force from the ground *plus* the factored live load surcharge calculated *previously*.

C.5.5 Bearing Resistance

Step eight of the MSE Wall design consists of checking the bearing resistance beneath the wall. According to Earth Retaining Structures:

> *Because of* the flexibility of MSE walls and the inability of the flexible reinforcement to transmit moment, a uniform base pressure distribution is assumed over an equivalent footing width. Unlike the bearing resistance check for *Cast-in-place* walls founded on rock, the assumption of a uniform base pressure is used for MSE walls founded on rock. The effect of eccentricity, load inclination, and live load surcharges must be included in this check. Effects of live load surcharges are included because they increase the loading on *the* foundation.

The factored bearing resistance (q_r) is given as:

$$q_r = \varphi q_n \qquad \text{Equation C-13}$$

Where,

φ = Resistance factor *(see Chapter 9)*
q_n = Nominal bearing resistance *(see Chapter 15)*

To check whether the bearing resistance of the MSE wall is adequate, the q_r computed *previously* is compared against the following criterion:

$$q_r \geq q_{uniform}$$ Equation C-14

Where,

$q_{uniform}$ = Vertical stress for walls on soil foundations, which is calculated assuming a uniform distribution of pressure over an effective base width (*B'*)

$$q_{uniform} = \frac{V_{TOT}}{B'}$$ Equation C-15

Where,

V_{TOT} = Sum of all factored vertical forces acting at the base of the wall (e.g., weight of reinforced fill, live and dead load surcharges, *etc.*)

and,

$$B' = B - 2e$$ Equation C-16

Where,

B = Length of reinforcement

e = Eccentricity determined from *Equation C-6*, however, for bearing resistance calculations, X_o is defined as:

$$X_o = \frac{(M_{VTOT} - M_{HTOT})}{V_{TOT}}$$ Equation C-17

Where,

M_{VTOT} = Resisting moment due to factored total vertical load based on earth pressure and live load surcharge calculated about the toe of the wall

M_{HTOT} = Driving moment due to factored lateral load based on earth pressure and live load surcharge calculated about the toe of the wall

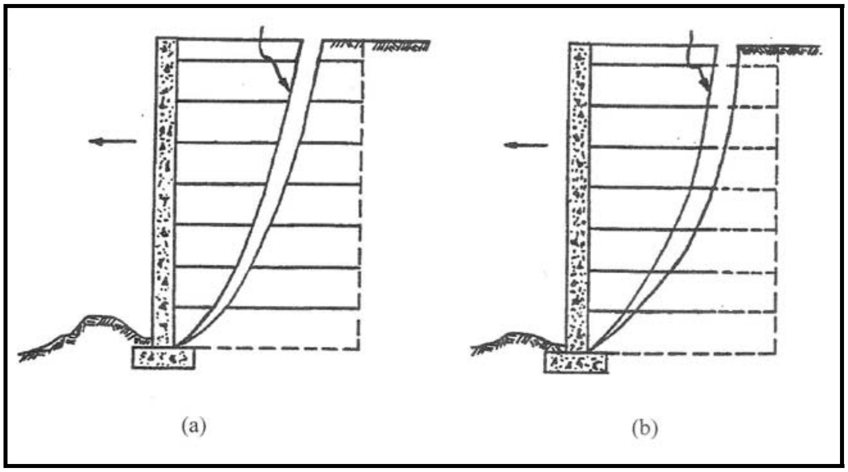
C.6 INTERNAL STABILITY

The internal stability analysis is the ninth step to the thirteenth step of the design process provided in Chapter 18. These analyses are normally performed by the MSE wall supplier or manufacturer. According to Earth Retaining Structures:

"To be internally stable, the MSE structure must be coherent and self supporting under the action of its own weight and any externally applied forces. This is accomplished through stress transfer from the soil to the reinforcement. This interaction between the soil and reinforcement improves the tensile properties and creates a composite material with the following characteristics:

- Stress transfer between the soil and reinforcement takes place continuously along the reinforcement; and
- Reinforcements are distributed throughout the soil mass with a degree of regularity and must not be localized.

Figure *C-7* illustrates the internal failure mechanisms for MSE walls. At each reinforcement level, the reinforcement must be sized and spaced to preclude rupture under the stress it is required to carry and to prevent pullout from the soil mass. The process of sizing and designing to preclude internal failure, therefore, consists of determining the maximum developed tension forces in the reinforcements (i.e., maximum load), the location of the load against the resistance provided by the reinforcements both in pullout and tension.



(a) Tension Failure (b) Pullout Failure

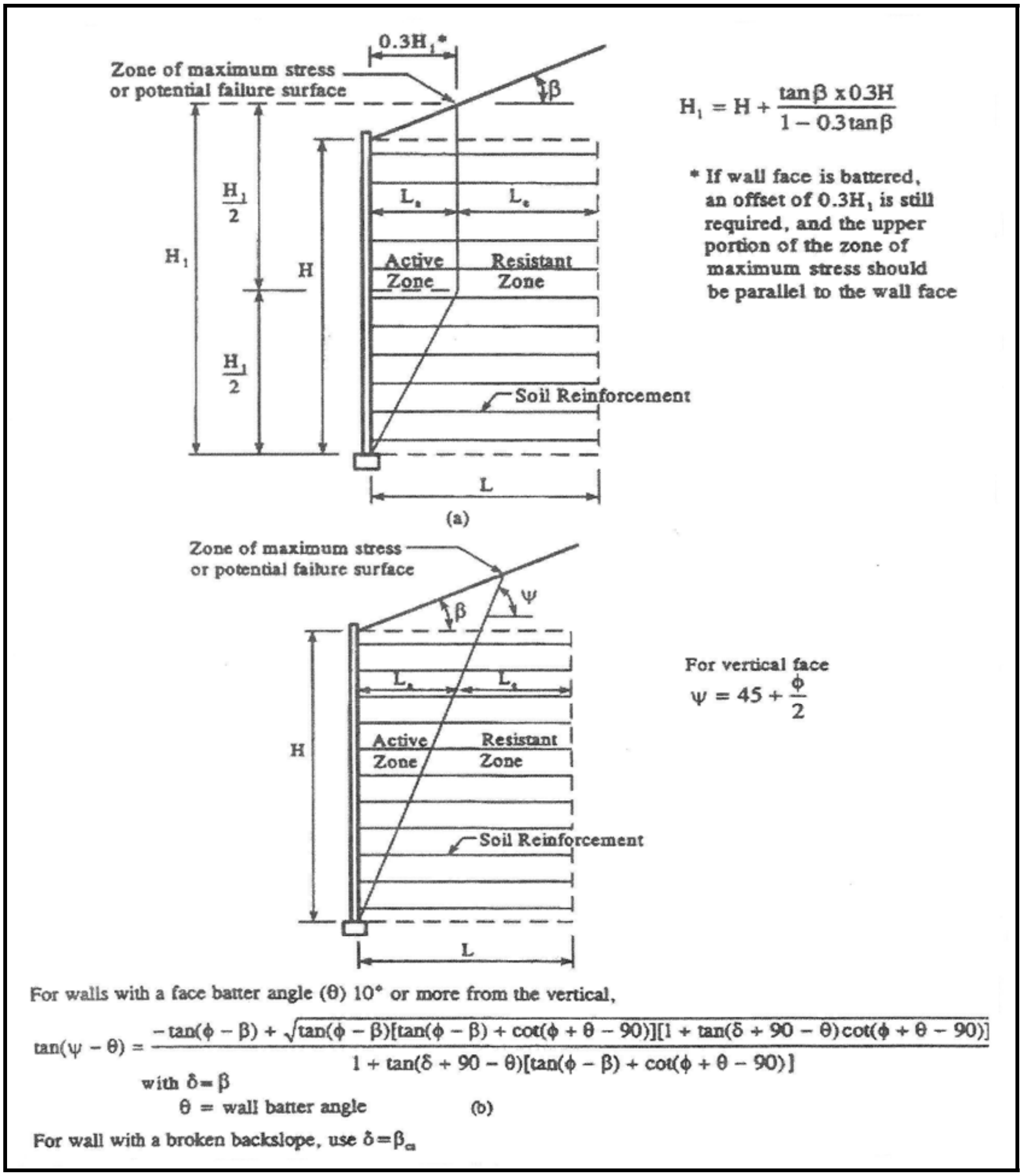
Figure C-7, MSE Wall Internal Failure Mechanisms (Earth Retaining Structures – June 2008)

C.6.1 Critical Failure Surface Location

The location and shape of the theoretical critical failure surface is dependent on the type of reinforcement. Therefore, at this stage of the design, the reinforcement type must be selected first, categorized (i.e., extensible or inextensible) and, then, the potential critical failure surface can be calculated.

When inextensible reinforcements are used, the soil deforms more than the reinforcement. Therefore, the soil strength in this case is measured at low strain. The critical failure surface for this reinforcement type is determined by dividing the reinforced zone into active and resistant zones with a bilinear failure surface as shown in Figure *C-8a*.

When extensible reinforcements are used, the reinforcement deforms more than the soil. Therefore, it is assumed that the shear strength of the reinforced fill is fully mobilized (residual strength) and active lateral earth pressures are developed. As a result, the critical failure surface for both horizontal and sloping backfill conditions are represented by the Rankine active earth pressure zone as shown in Figure *C-8b*.



(a) Inextensible reinforcements (b) Extensible reinforcements

Figure C-8, Potential Failure Surface Location for Internal Stability of MSE Walls (Earth Retaining Structures – June 2008)

C.6.2 Factored Horizontal Stress

The purpose of this design step is to calculate the maximum factored horizontal stress. It is specifically noted that load factors are typically applied to unfactored loads, not to an unfactored stress (as it is in Equation *C-18*) below. The AASHTO (2007) LRFD code, however, applies the "load" factor to the unfactored "stress" for this particular design calculation. The factored horizontal stress (σ_H) at each reinforcement is based on *Equation C-18*.

$$\sigma_H = \gamma_P \left(\sigma_V \boldsymbol{k}_r + \Delta\sigma_H\right) \qquad \text{Equation C-18}$$

Where,

γ_P = Maximum load factor for vertical earth pressure (EV) (*see Chapter 8*)

k_r = Lateral earth pressure coefficient from Equation C-20

σ_V = Pressure due to resultant of gravity forces from soil *unit* weight within and immediately above the reinforced wall backfill, and any surcharge loads present

$\Delta\sigma_H$ = Horizontal stress at reinforcement level resulting in a concentrated horizontal surcharge load (refer to Article 11 of AASHTO)

Research studies have indicated that the maximum tensile force is primarily related to the type of reinforcement in the MSE mass, which, in turn, is a function of the modulus extensibility, and density of reinforcement. Based on this research, a relationship between the type of reinforcement and the overburden stress has been developed and is shown in Figure *C-9*.

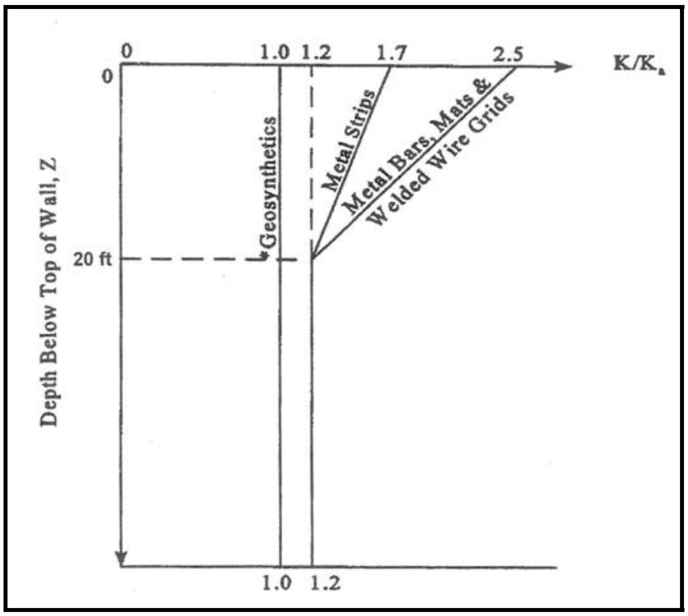


Figure C-9, Variation of the Coefficient of Lateral Stress Ratio (Earth Retaining Structures – June 2008)

Figure *C-9* was prepared by back *calculation* of the lateral stress ratio from available field data where stresses in the reinforcements have been measured and normalized as a function of an active earth pressure coefficient. The lines shown on this figure correspond to usual values representative of the specific reinforcement systems that are known to give satisfactory results, assuming that the vertical stress is equal to the weight of the overburden.

For a vertical wall face (i.e., batters less than 8 degrees from vertical), the active earth pressure coefficient (K_a) is determined from Equation *C-19*. For wall face batters equal to or greater than 8 degrees from the vertical, the simplified form of the Coulomb equation as presented in AASHTO (2007) and shown *in Equation C-20, is used to calculate active earth pressure.*

$$K_a = \tan^2\left(45 - \frac{\varphi_r}{2}\right) \qquad \text{Equation C-19}$$

$$K_a = \frac{\sin^2(\theta + \varphi_r')}{\sin^3\theta\left(1 + \frac{\sin\varphi_r'}{\sin\theta}\right)^2} \qquad \text{Equation C-20}$$

Where,

θ = Inclination of the back of the facing as measured from the horizontal starting in front of the wall

φ_r' = Friction angle of reinforced fill

The value of K_a in the reinforced soil mass is assumed to be independent of all external loads, even sloping fills. If testing of the site-specific select backfill is not available, the value of φ_r used to compute the horizontal stress within the reinforced soil mass should not exceed 34°.

Once the value of K_a is known, the lateral earth pressure coefficient (k_r) that is used to compute σ_H at each reinforcement level is calculated as:

$$k_r = \left(\frac{K}{K_a}\right)K_a \qquad \text{Equation C-21}$$

Where,

K/K_a = From Figure *C-9*

K_a = From Equation *C-19 or C-20*

If present, surcharge load should be added into the estimation of σ_V. For sloping soil surfaces above the MSE wall section, the actual surcharge is replaced by a uniform surcharge equal to half of the height of the slope at the back of the reinforcements. For cases where concentrated vertical loads occur, refer to *Earth Retaining Structures – June 2008* for computation of σ_V.

C.6.3 Maximum Factored Tensile Stress

The maximum tension in each reinforcement layer per unit width of wall (T_{max}) based on the reinforcement vertical spacing (S_V) is calculated as:

$$T_{\max} = \sigma_H S_V \qquad \text{Equation C-22}$$

Where,

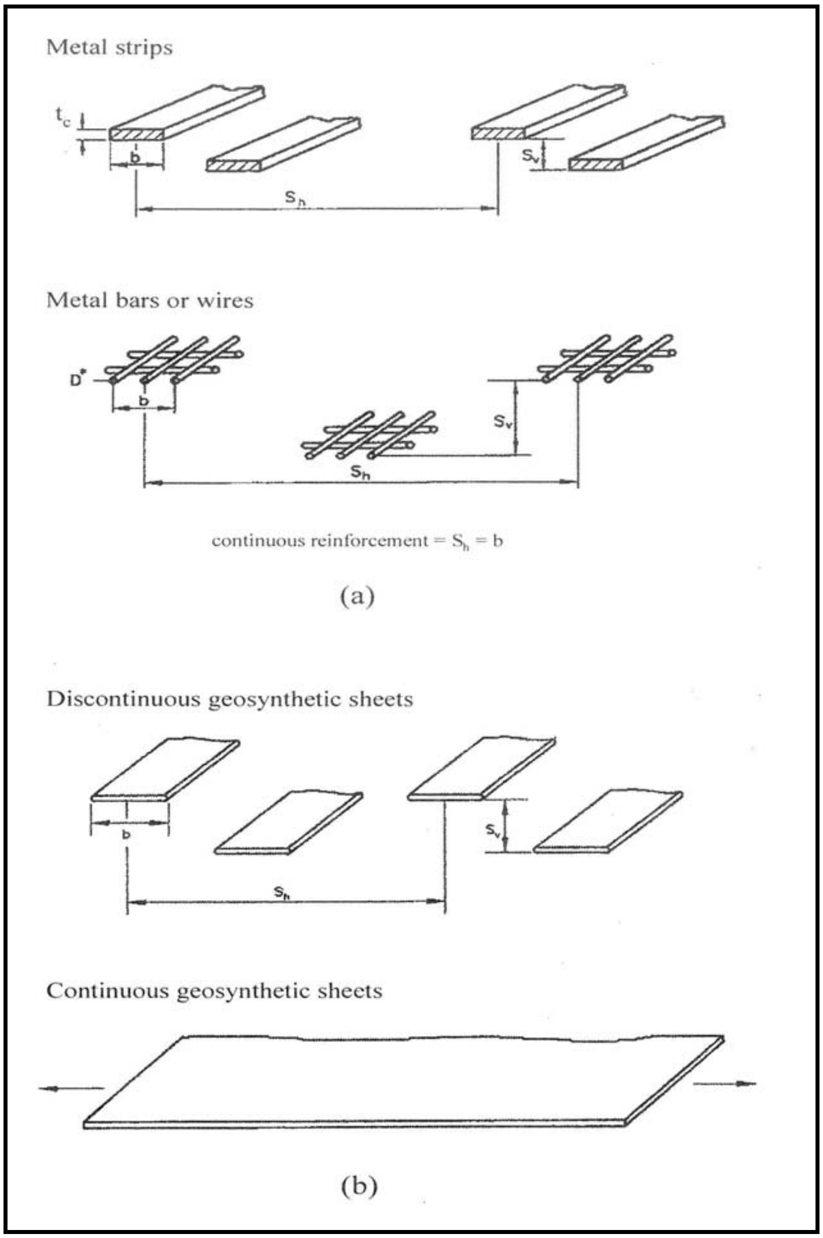
σ_H = Factored horizontal load calculated using Equation *C-18*.

T_{max} may also be calculated at each level for discrete reinforcements (metal strips, bar mats, grids, etc.) per a defined unit width of reinforcement as:

$$T_{max-R} = \frac{\sigma_H S_V}{R_c} \qquad \text{Equation C-23}$$

Where,

R_c = Reinforcement coverage ratio (b/S_h) (*see Figure C-10*)
b = Gross width of the reinforcing element
S_h = Center-to-center horizontal spacing between reinforcements



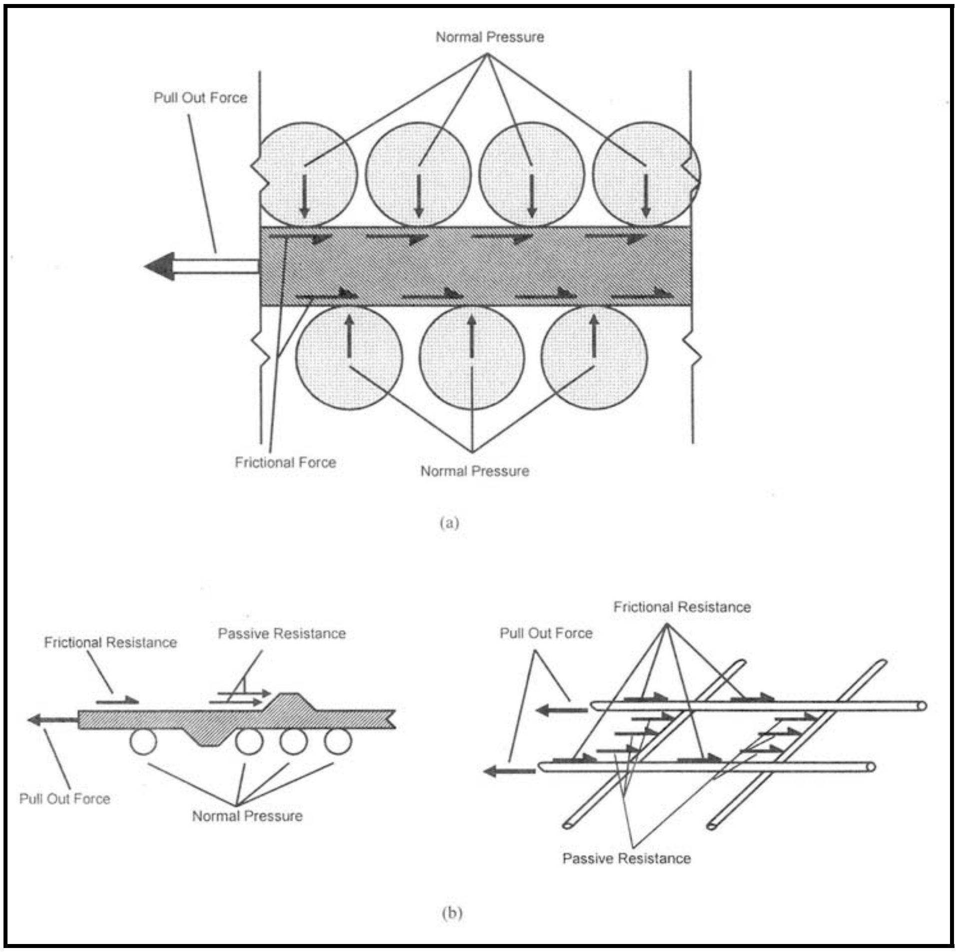
(a) Metal reinforcement (b) Geosynthetic reinforcement

Figure C-10, Definitions of b, S_h and S_V
(Earth Retaining Structures – June 2008)

C.6.4 Reinforcement Pullout Resistance

The purpose of this design step is to check the pullout resistance of the reinforcements. The resistance develops after the stress transfer between the soil and reinforcement takes place, *stress transfer* occurs through two mechanisms;

1. Friction along the soil-reinforcement interface (*see Figure C-11a)*
2. Passive soil resistance or lateral bearing capacity developed along the transverse sections of the reinforcement (*see Figure C-11b)*



(a) By friction (b) By passive resistance

Figure C-11, Mechanisms of Pullout Resistance
(Earth Retaining Structures – June 2008)

Stresses are transferred between soil and reinforcement by friction and/or passive resistance depending on reinforcement geometry.

Friction develops at locations where there is a relative shear displacement and corresponding shear stress between soil and reinforcement surface. Reinforcing elements where friction is important should be aligned with the direction of soil reinforcement relative movement. Examples of such reinforcing elements are steel strips, longitudinal bars in grids, geotextile, and some geogrid layers.

Passive Resistance occurs through the development of bearing type stresses on "transverse" reinforcement surfaces normal to the direction of soil reinforcement relative movement. Passive resistance is generally considered to be the primary interaction for rigid geogrids, bar mat, and wire mesh reinforcements. The transverse ridges on "ribbed" strip reinforcements also provide some passive resistance.

The contribution of each transfer mechanism for a particular reinforcement will depend on the roughness of the surface (skin friction), normal effective stress, grid opening dimensions, thickness of the transverse members, and elongation characteristics of the reinforcement. Equally important for interaction development are the soil characteristics, including grain size and grain size distribution, particle shape, density, water content, cohesion, and stiffness.

The primary function of reinforcement is to restrain soil deformations. In doing so, stresses are transferred from the soil to the reinforcement. These stresses are carried by the reinforcement in two ways: in tension or in shear and bending.

The unfactored pullout resistance (P_r) of the reinforcement per unit width of reinforcement is estimated as:

$$\boldsymbol{P_r = F^* \alpha \sigma_v} \qquad \text{Equation C-24}$$

Parameters in Equation *C-24* are defined in Equation *C-26*.

In this design step, the reinforcement pullout resistance is evaluated at each reinforcement level of the MSE wall. The required total length for reinforcement to generate *sufficient* pullout resistance for each level is calculated and then compared against the total reinforcement length initially estimated *previously*. The initially estimated total reinforcement length may have to be adjusted based on the required length calculated in this step.

The total length of reinforcement (L) required for internal stability is determined as:

$$\boldsymbol{L = L_e + L_a} \qquad \text{Equation C-25}$$

Where,

L_e = Required length of reinforcement in resisting zone (i.e., beyond the potential failure surface)

L_a = Remainder length of reinforcement

C.6.4.1 Estimating L_e

The length of reinforcement in the resisting zone (L_e) is determined using the following equation:

$$L_e \geq \frac{T_{max}}{\varphi F^* \alpha \sigma_v C R_c}$$ Equation C-26

Where,

T_{max} = Maximum factored tensile load in the reinforcement (calculated *by Equation C-22)*

φ = Resistance factor for reinforcement pullout (*see Chapter 9*)

F* = Pullout friction factor (discussed below)

α = Scale effect correction factor (discussed below)

σ_v = Unfactored vertical stress at the reinforcement level in the resistance zone

C = Overall reinforcement surface area geometry factor (2 for strip, grid and sheet-type reinforcement)

R_c = Reinforcement coverage ratio (*see Equation C-23 and Figure C-10)*

C.6.4.2 Correction Factor (α)

The correction factor (α) depends primarily upon the strain softening of the compacted granular backfill material, and the extensibility, and the length of the reinforcement. Typical values of α based on reinforcement type are presented in Table *C-7*. For inextensible reinforcement, α is approximately 1, but it can be substantially smaller than 1 for extensible reinforcements. The α factor can be obtained from pullout tests on reinforcements with different lengths or derived using analytical or numerical load transfer models which have been "calibrated" through numerical test simulations. In the absence of test data, *the values included in Table C-7 should be used for geogrids and geotextiles.*

Table C-7, Typical Values of α
(Earth Retaining Structures – June 2008)

Reinforcement Type	α
All steel reinforcements	1.0
Geogrids	0.8
Geotextiles	0.6

C.6.4.3 Pullout Friction Factor (F*)

The pullout friction factor can be obtained most accurately from laboratory or field pullout tests performed with the specific material to be used on the project (i.e., select backfill and reinforcement). Alternatively, F* can be derived from empirical or theoretical relationships developed for each soil-reinforcement interaction mechanism and provided by the reinforcement supplier. For any reinforcement, F* can be estimated using the general equation:

$$F^* = F_q \alpha_\beta + \tan \rho$$ Equation C-27

Where,

F_q = The embedment (or surcharge) bearing capacity factor

α_β = A bearing factor for passive resistance which is based on the thickness per unit width of the bearing member

ρ = The soil-reinforcement interaction friction angle

Equation *C-27* represents systems that have both the frictional and passive resistance components of the pullout resistance. In certain systems, however, one component is much smaller than the other and can be neglected for practical purposes.

In absence of site-specific pullout test data, it is reasonable to use these semi-empirical relationships in conjunction with the standard specifications for backfill to provide a conservative evaluation of pullout resistance.

For steel ribbed reinforcement, F* is commonly estimated as:

$$F^* = \tan\rho = 1.2 + \log C_u \qquad \text{Equation C-28}$$

at the top of the structure =2.0 maximum

$$F^* = \tan\varphi_r \qquad \text{Equation C-29}$$

at a depth of 20 feet and below

Where,

ρ = Interface friction angle mobilized along the reinforcement

C_u = Uniformity coefficient of the backfill (*see Chapter 6*)

If the specific C_u for the wall backfill is unknown during design, a C_u of 4 should be assumed (i.e., F* = 1.8 at the top of the wall), for backfill meeting the requirements *previously provided.*

For steel grid reinforcements with transverse spacing (S_t) $\geq$ 6 inches, F* is a function of a bearing or embedment factor (F_q), applied over the contributing bearing factor (α_β), as follows:

$$F^* = F_q\alpha_\beta = 40\alpha_\beta = 40\left(\frac{t}{2S_t}\right) = 20\left(\frac{t}{S_t}\right) \qquad \text{Equation C-30}$$

at the top of the structure

$$F^* = F_q\alpha_\beta = 20\alpha_\beta = 20\left(\frac{t}{2S_t}\right) = 10\left(\frac{t}{S_t}\right) \qquad \text{Equation C-31}$$

at a depth of 20 feet and below

Where,

t = The thickness of the transverse bar

S_t = *The distance between individual bars in steel grid reinforcement and* shall be uniform throughout the length of the reinforcement, rather than having transverse grid members concentrated only in the resistance zone *(see Figure C-12)*

For geosynthetic (i.e., geogrid and geotextile) sheet reinforcement, the pullout resistance is based on a reduction in the available soil friction with the reduction factor often referred to as an interaction factor (C_i). In the absence of test data, the F* value for geosynthetic reinforcement should conservatively be estimated as:

$$F^* = 0.67\tan \acute{\omega} \qquad \text{Equation C-32}$$

Where,

$\acute{\omega}$ = *Wall fill peak friction angle*

When used in the above relationship, $\acute{\omega}$ is the peak friction angle of the soil which, for MSE walls using select granular backfill, is taken as 34 degrees unless project specific test data substantiates higher values.

The relationship between F* and depth below the top of the wall for different reinforcement types is summarized in Figure *C-12*.

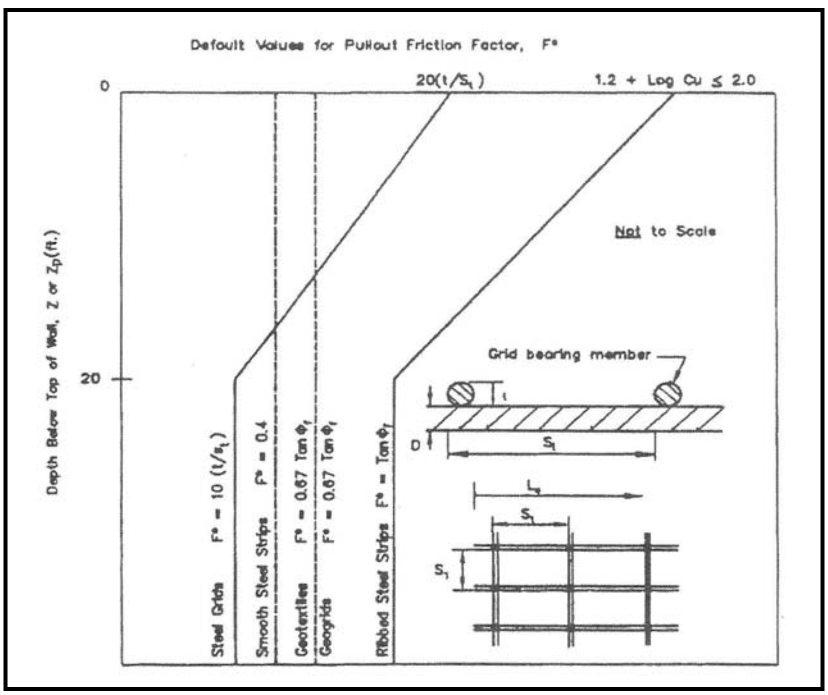


Figure C-12, Typical Values of F*
(Earth Retaining Structures – June 2008)

C.6.4.4 Estimating L_a

The L_a is obtained from Figure *C-8* for simple structures not supporting concentrated external loads such as bridge abutments. Based on this figure, the following relationships can be obtained for L_a:

- For MSE walls with extensible reinforcement, vertical face, and horizontal backfill:

$$L_a = (H - Z)\tan\left(45 - \frac{\varphi_r'}{2}\right) \qquad \text{Equation C-33}$$

Where,

Z = Depth to the reinforcement level

- For walls with inextensible reinforcement form the base up to H/2:

$$L_a = 0.6(H - Z) \qquad \text{Equation C-34}$$

- For the upper half of a wall with inextensible reinforcements:

$$L_a = 0.3H \qquad \text{Equation C-35}$$

For ease of construction, based on the maximum total length required, a final uniform reinforcement length is commonly chosen. However, if internal stability controls the length, it could be varied from the base, increasing with the height of the wall to the maximum length requirement based on a combination of internal and maximum external stability requirements.

C.6.5 Long-Term Reinforcement Design Strength

In this design step, the maximum factored tensile stress in each reinforcement layer (T_{max}, *determined previously*) is compared to the nominal long-term reinforcement design strength as presented below:

$$T_{max} \leq \varphi R_c T_{al} \qquad \text{Equation C-36}$$

Where,

φ = Resistance factor for tensile resistance (*see Chapter 9*)
R_c = Reinforcement coverage ratio as defined in Equation *C-23 and Figure C-10*)
T_{al} =Nominal long-term reinforcement design strength

The nominal long-term reinforcement design strength (T_{al}) for LRFD is computed for inextensible and extensible reinforcements as presented *in the following sections.*

C.6.5.1 T_{al} for Inextensible Reinforcements

The nominal long-term design strength of inextensible reinforcement is provided below:

$$T_{al} = \frac{A_c F_y}{b} \qquad \text{Equation C-37}$$

Where,

F_y = Minimum yield strength of steel
b = Unit width of sheet, grid, bar or mat
A_c = Design cross sectional area corrected for corrosion loss

The lower resistance factor of 0.65 (*see Chapter 9*) for grid reinforcement (as compared to a resistance factor of 0.75 (*see Chapter 9*) for strip reinforcement) accounts for the greater potential for local overstress due to load nonuniformities for steel grids than for steel strips or bars.

A_c for strips is determined as:

$$A_c = bt_c = b(t_n - t_s)$$ Equation C-38

Where,

b = Unit width of sheet, grid, bar or mat
t_c = Thickness at end of design life (see Figure *C-13*)
t_n = Thickness at end of construction
t_s = Sacrificial thickness of metal expected to be lost by uniform corrosion during the service life of the structure

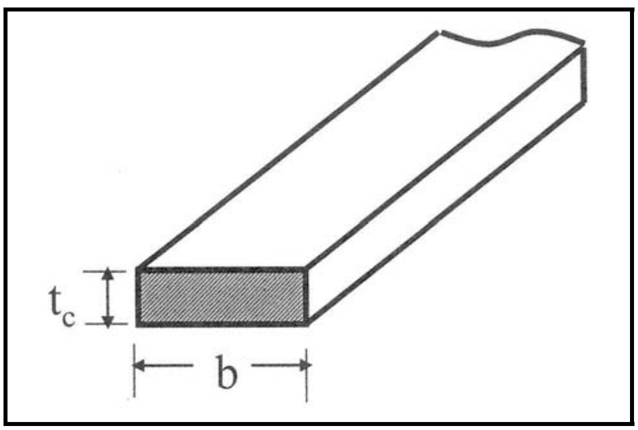


Figure C-13, Cross Section Area for Strips (Earth Retaining Structures – June 2008)

When estimating t_s, it may be assumed that equal loss occurs from the top and bottom of the strip.

A_c for bars is determined as:

$$A_c = N_b \left(\frac{\pi D^{*2}}{4} \right)$$ Equation C-39

Where,

N_b = Number of bars per unit width b
D* = Bar diameter after corrosion loss (Figure *C-14*)

When estimating D*, it may be assumed that corrosion losses occur uniformly over the area of the bar.

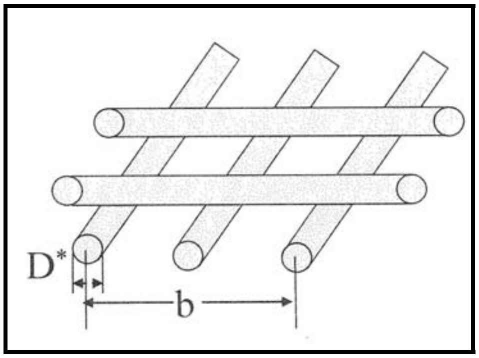


Figure C-14, Cross Section Area for Bars
(Earth Retaining Structures – June 2008)

C.6.5.2 Corrosion Rates

The corrosion rates presented below are suitable for conservative design. These rates assume a mildly corrosive backfill material having the controlled electrochemical property limits that are discussed *previously*.

<u>Corrosion Rates</u> – mildly corrosive backfill

For corrosion of galvanization on each side

- 0.58 mil./year/side (first 2 years)
- 0.16 mil./year/side (thereafter)

For corrosion of residual carbon steel on each side

- 0.47 mil./year/side (after zinc depletion)

Based on these rates, complete corrosion of galvanization with the minimum required thickness of 3.4 millimeters (*mil.*) (AASHTO M 111) is estimated to occur during the first 16 years and a carbon steel thickness or diameter loss of 0.055 inches to 0.08 inches would be anticipated over the remaining 100-year design life, respectively. The designer of an MSE structure should also consider the potential for changes in the reinforced backfill environment during the structure's service life. In certain parts of *South Carolina*, it can be expected that deicing salts might cause such an environment change. For this problem, the depth of chloride infiltration and concentration are of concern.

For permanent structures directly supporting roadways exposed to deicing salts, limited data indicate that the upper 8 feet of the reinforced backfill (as measured from the roadway surface) are affected by higher corrosion rates not presently defined. Under these conditions, it is recommended that a 30 mil (minimum) geomembrane be placed below the road base and tied into a drainage system to mitigate the penetration of the deicing salts in lieu of higher corrosion rates.

C.6.5.3 T_{al} for Extensible Reinforcements

The nominal long-term design strength of extensible reinforcement is provided below:

$$T_{al} = \frac{T_{ult}}{RF} \quad \text{Equation C-40}$$

Where,

T_{ult} = Minimum average roll value ultimate tensile strength
RF = Combined strength reduction factor to account for potential long-term degradation due to installation damage, creep and chemical aging

$$RF = RF_{ID} RF_{CR} RF_{D} \quad \text{Equation C-41}$$

Where,

RF_{ID} = Strength reduction factor to account for installation damage to reinforcement
RF_{CR} = Strength reduction factor to prevent long-term creep rupture of reinforcement
RF_D = Strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation

According to AASHTO (2007) values of RF_{ID}, RF_{CR}, RF_D shall be determined from product specific test results."

C.6.5.4 T_{ac} for Connection Strength

Allowable connection loads, T_{ac}, for various levels of soil reinforcement shall be determined at the anticipated vertical confining pressure at the wall face between the facing blocks. The height of the wall above the wall interface shall be used to compute the vertical confining pressure for walls with a nominal batter of 8 degrees or less. For walls with a nominal batter of more than 8 degrees, the vertical confining pressure is limited to the lesser of the height obtained by the "Hinge Height Method" as described in AASHTO or the height of the wall above the interface.

For metallic soil reinforcement, the factored tensile load applied to the reinforcement connection at the wall face (T_{ac}), shall be equal to the maximum factored reinforcement tension, T_{max}, as determined previously, for all wall systems regardless of the facing or reinforcement (bar or grid) type.

For geosynthetic soil reinforcement, check that the static allowable connection load, T_{ac}, multiplied by the reinforcement coverage ratio, R_c, (R_c=1.0 for geosynthetic sheet type reinforcement, i.e. geogrid) shall meet or exceed the maximum applied load to the soil reinforcement connection, T_o, at each level of reinforcement placement at the wall facing. The static allowable connection load, T_{ac}, for geosynthetic reinforcement is determined using the following design methodology.

The allowable connection strength, T_{ac}, per unit width of reinforcement at the connection shall be computed by determining the long-term connection strength through laboratory connection strength testing. The long-term connection strength may be determined by either a long-term connection test or by a quick load connection test in accordance with the procedures outlined in Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines FHWA NHI-00-043, March 2001. When the connection system has more than one type of material or component (i.e. connector, geogrid), the long-term connection strength shall be determined from the material or component that produces the lowest connection strength. The allowable connection strength, T_{ac}, shall not exceed the allowable tensile load, T_a, of the soil reinforcement multiplied by the connection coverage ratio, C_c. The connection coverage ratio, C_c, is defined as the width of the connectors within a block width divided by the width of the block.

When a long-term connection test is performed, the allowable connection strength is determined by the following equation.

$$T_{ac} = \frac{T_{ult} CR_{cr}}{RF_{DC}} \leq T_a C_c \qquad \text{Equation C-42}$$

When the quick load connection test is performed, the allowable connection strength is determined by the following equation.

$$T_{ac} = \frac{T_{ult} CR_{ult}}{RF_{CRDC}} \leq T_a C_c \qquad \text{Equation C-43}$$

C.6.5.5 Reduction Factor RF_{ID}

RF_{ID} can range from 1.05 to 3.0 depending on backfill gradation and product mass per unit weight. Even with product specific test results, the minimum reduction factor shall be 1.1 to account for testing uncertainties. The placement and compaction of the backfill material against the geosynthetic reinforcement may reduce its tensile strength. The level of damage for each geosynthetic reinforcement is variable and is a function of the weight and type of the construction equipment and the type of geosynthetic material. The installation damage is also influenced by the lift thickness and type of soil present on either side of the reinforcement. Where granular and angular soils are used for backfill, the damage is more severe than when softer, finer, soils are used. For a more detailed explanation of the RF_{ID} factor, see Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, FHWA-NHI-00-044.

C.6.5.6 Reduction Factor RF_{CR}

RF_{CR} is obtained from long-term laboratory creep testing as detailed in Elias et al. (2001). This reduction factor is required to limit the load in the reinforcement, to a level known as the creep limit, that will preclude creep rupture over the life of the structure. Creep in itself does not degrade the strength of the polymer. Creep testing is essentially a constant load test on multiple product samples, loaded to various percentages of the ultimate product load, for periods of up to 10,000 hours. The creep reduction factor is the ratio of the ultimate load to the extrapolated maximum sustainable load (i.e., creep limit) within the design life of the structure (e.g., several years for temporary structures (maximum 5 years) or 100 years for permanent structures). Typical reduction factors as a function of polymer type are indicated in Table *C-8*.

Table C-8, Creep Reduction Factors
(Earth Retaining Structures – June 2008)

Polymer Type	RF_{CR}
Polyester	1.6 to 2.5
Polypropylene	4.0 to 5.0
Polyethylene	2.6 to 5.0

If no product specific creep reduction factors are provided, then, the maximum creep reduction factor for a specific polymer shall be used. If the polymer is unknown, then, an RF_{CR} of 5.0 shall be used.

C.6.5.7 Reduction Factor RF_D

RF_D is dependent on the susceptibility of the geosynthetic to be attacked by microorganisms, chemicals, thermal oxidation, hydrolysis and stress cracking and can vary typically from 1.1 to 2.0. Even with product specific tests results, the minimum reduction factor shall be 1.1. A protocol for testing to obtain this reduction factor have been described in Elias et al. (1997).

C.6.5.8 Connection Strength Reduction Factors CR_{CR} and CR_{ult}

The long-term connection test strength reduction factor, CR_{cr}, accounts for the reduced connection capacity at the end of the design life of the MSE wall structure due to connection failure and also includes reductions in material strength due to creep of the failed connection component. The connection strength reduction factors shall be certified based on the method of long-term connection testing selected. Either a long-term connection test or a quick load connection test may be used to certify the connection strength reduction factors in accordance with the procedures outlined in Appendix "A.3" of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines. When a connection system has more than one type of material or component (i.e. connector, geogrid), the connection strength reduction shall be determined from the material or component that produces the lowest long-term connection strength.

If a long-term connection test is used, the long-term strength reduction factor, CR_{CR}, is computed by the following equation:

$$CR_{CR} = \frac{T_{crc}}{T_{lot}} \qquad \text{Equation C-44}$$

Where,

T_{lot} = Ultimate wide width tensile strength of the soil reinforcement material lot used for long-term connection strength testing.

T_{crc} = Ultimate connection strength, T_{ult}, at the end of a 1,000 hour connection test for each normal load is then extrapolated to the design life ultimate connection strength.

The quick load connection test strength reduction factor, CR_{ult}, accounts for the reduced connection capacity due to connection failure, without taking into account the reduction in strength due to creep of the failed connection component. If a quick load connection test is used, the quick load strength reduction factor, CR_{ult}, is computed by the following equation:

$$CR_{ult} = \frac{T_{ultconn}}{T_{lot}} \qquad \text{Equation C-45}$$

Where,

T_{lot} = Ultimate wide width tensile strength of the soil reinforcement material lot used for quick load connection strength testing

$T_{ultconn}$ = Ultimate connection strength for each peak connection load at each normal load

C.6.5.9 Creep (RF_{CRC}), Durability (RF_{DC}), and Combined Creep/Durability (RF_{CRDC}) Reduction Factors at Wall Facing Connections

The individual reduction factors for creep, RF_{CRC}, and durability (RF_{DC}) at the connection shall be documented in accordance with the MSE wall environmental conditions at the connection location. The reinforcement manufacturer shall certify the individual reduction factors for creep (RF_{CRC}) in accordance with the same procedures listed for creep reduction factor (RF_{CR}) and for durability (RF_{DC}) in accordance with the same procedures listed for individual reduction factor for durability (RF_D).

When long-term connection testing is performed and sufficient documentation is not provided for the durability reduction factor, RF_{DC}, the default durability reduction factor, RF_D, shall be used. When only the quick load connection test is performed and sufficient documentation is not provided for the combined reduction factors for creep and durability (RF_{CDRC}), the total default connection factor, $RF_{CDefault}$, shall be 6.0. The combined reduction factor for creep and durability (RF_{CRDC}) is computed by the following equation.

$$RF_{CRDC} = RF_{CRC} RF_{DC} \qquad \text{Equation C-46}$$

The individual reduction factors for creep, RF_{CRC}, and for durability, RF_{DC}, shall be the certified reduction factors for the environmental conditions that exist at the connection location.

C.7 OVERALL STABILITY

The overall (global) stability is typically determined by the geotechnical engineer-of-record. This stability can be determined using classical slope stability analyses (see Chapter 17). The failure surfaces may be circular or non-circular and both should be checked. Typically, it is assumed that the failure surface does not pass through the reinforced mass of the MSE structure; therefore, the MSE structure is given strength parameters greater than the retained and foundation soils to prevent the failure plane from passing through the reinforced soil mass. Overall stability analyses are performed for the Service limit state and are normally performed once the initial estimate of the reinforcement length is determined from Step 3 (see Section C.4 above). The results of the overall stability analysis can and do affect the reinforcement length used in the design. It should be noted that it is assumed that all MSE walls are free draining and that pore water pressures are not allowed to build up behind the wall.

Prior to submission of the final design plans, a compound global stability analysis shall be performed. As defined in Chapter 18, a compound stability analysis examines failure surfaces that pass through either the retained fill and reinforced soil mass to exit through the MSE wall face or that pass through the retained fill, reinforced soil mass and the foundation soil to exit beyond the toe of the MSE wall. The actual strength parameters that the reinforced soil mass is based on shall be used in the analysis. These analyses can only be performed once a specific MSE wall type is selected. This analysis may be performed by either the geotechnical engineer-of-record or by the MSE wall supplier.

The resistance factors (φ) for global stability analyses are provided in Chapter 9. MSE wall structures are considered Flexible Gravity Retaining Walls.

C.8 WALL DISPLACEMENT ESTIMATION

MSE wall structures can move both vertically and horizontally due to static and seismic loads. The movements caused by static loads are discussed here. See Chapter 14 for guidance on seismically induced movements. Vertical movements (settlement) should be determined using the procedures outlined in Chapter 17. In conditions where the reinforced soil mass will settle more than the face, the reinforcement should be placed on a sloping fill surface, which is higher at the backend of the reinforcement to compensate for the greater vertical movement in this area. The reinforcement connection strength shall be checked if there is any differential settlement between the MSE wall face and the rear of the reinforced soil mass. This differential settlement can induce additional stresses in the connections at the interface between the reinforcement and wall face materials.

Differential settlements perpendicular to the MSE wall facing (along the soil reinforcement) may occur at roadway widening projects. If this type of differential settlement exceeds a ratio of 1/10, the MSE wall suppliers shall be consulted to determine if further analyses are required. The values shown in Table C-9 shall be used as typical limiting differential settlement tolerances along the MSE wall facing for MSE wall structures with precast panel facings.

Table C-9, Limiting Differential Settlement for MSE Wall Systems with Precast Concrete Panel Facing

Panel Joint Width	Limiting Differential Settlement
3/4” *	1/100
1/2” *	1/200
1/8”*	1/300
Full Height Panel	1/500

* **Note:** Relatively square facing panels

MSE wall structures with modular concrete block facings are typically restricted to a limiting differential settlement of 1/200 along the MSE wall structure. Temporary MSE wall structures with welded wire mesh facing should be restricted to a limiting differential settlement along the MSE wall facing of 1/50.

Slip joints may be used to maintain MSE wall structures within acceptable differential settlement tolerances. When significant differential settlements are anticipated, ground improvement techniques such as surcharges and wick drains may be required to accelerate the consolidation settlement. Walls shall be designed for any temporary surcharge loading. When long-term settlements are accelerated during construction, temporary wall facings may be required during this accelerated settlement phase followed by installation of permanent facings after the required level of settlement is achieved.

MSE wall structures may experience lateral displacements during construction that may affect the MSE wall performance. These lateral displacements are a function of overall structure stiffness, compaction intensity, soil type, reinforcement length, slack in reinforcement-to-facing connections, and deformability of the facing system. The MSE wall system supplier is responsible for determining the construction batter that is required to meet the geometry requirements shown in the plans. Estimates of lateral wall displacements occurring during construction may be necessary during plan preparation to determine minimum clearances between the wall face and adjacent structures. Based on a reinforcement embedment ratio of 0.7H, the following equations may be used to make a rough estimate of probable lateral displacement, δ_{est} for inextensible and extensible reinforcement, respectively:

$$\delta_{est} = \frac{H}{250} \qquad \text{Equation C-47}$$

$$\delta_{est} = \frac{H}{75} \qquad \text{Equation C-48}$$

Where,

δ_{est} = Estimated lateral displacement (ft)
H = Height of wall (ft)

C.9 WALL DRAINAGE SYSTEM DESIGN

The following section is adopted directly from Earth Retaining Structures – June 2008 and is used with the permission of the US Department of Transportation, Federal Highway Administration. The italics are added to reflect additions or modifications to the selected text and to supply references to this Manual.

> A drain at the base of the wall immediately behind the wall facing as shown in Figure *C-15*, is normally used with an MSE wall. This drain primarily serves to collect surface water that has infiltrated behind the facing and transports *the water* to an outlet. Outlet may be via weep holes, as shown in Figure *C-15*, or may be piped to a downslope outlet or to a storm sewer.

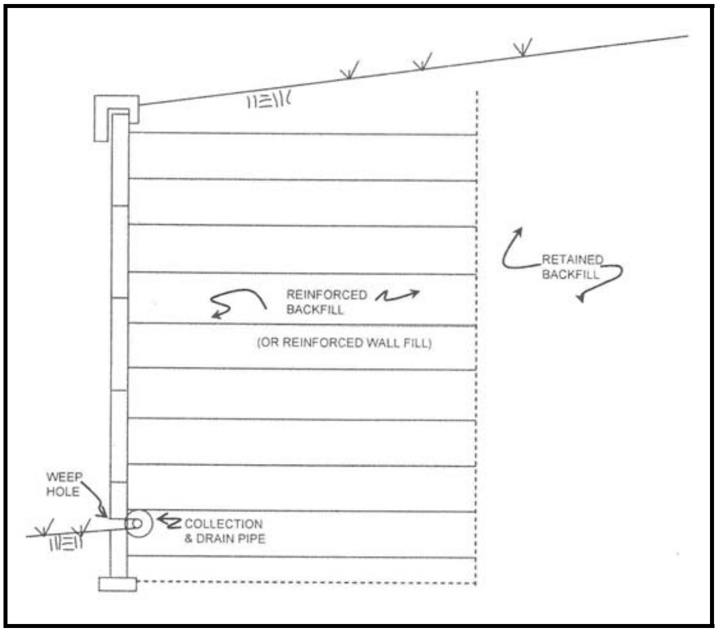


Figure C-15, Drain Immediately Behind MSE Wall Face (Earth Retaining Structures – June 2008)

This drain may also serve to drain the wall fill if it is relatively free-draining and the wall is used in a fill situation. A drain behind the wall backfill, as illustrated in Figure *C-16*, should be used when zones of soils (in-situ or retained backfill to wall) are not free-draining relative to one another. A drain is recommended to collect and divert groundwater from the reinforced fill for side hill construction, where in-situ soils are excavated to accommodate the reinforced fill volume. Additionally, a drain at the backcut or at the wall fill and retained backfill interface (for fill situations) is recommended, unless soil permeabilities, filtration, and water flow are analyzed to ensure system is free draining. Soil filtration and permeability requirements must be met between the two adjacent zones of (different) soils to prevent impeded flow.

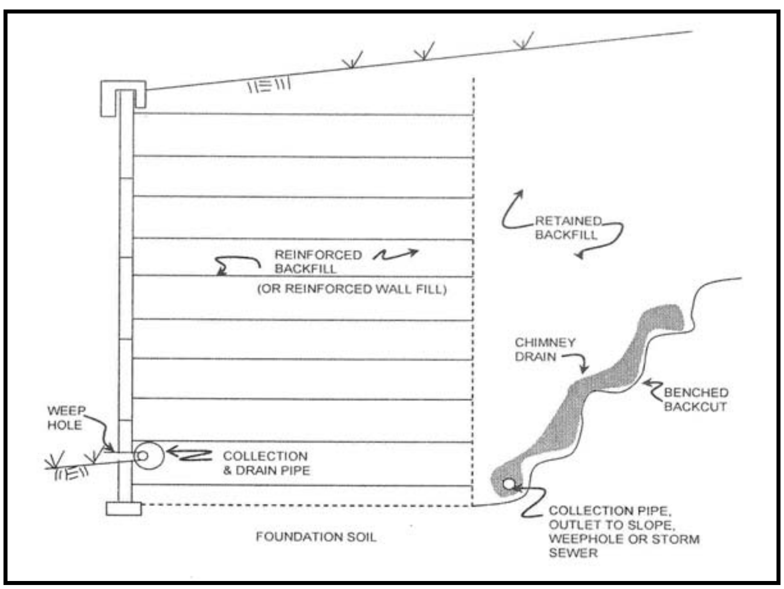


Figure C-16, Drain Behind the Wall Backfill (Earth Retaining Structures – June 2008)

A geomembrane barrier beneath the pavement structure, sloping to a drain, should be used where significant de-icing salts are used. The purpose of the barrier is to prevent, or minimize, the infiltration of water immediately behind the wall facing. A common detail is shown in Figure *C-17*.

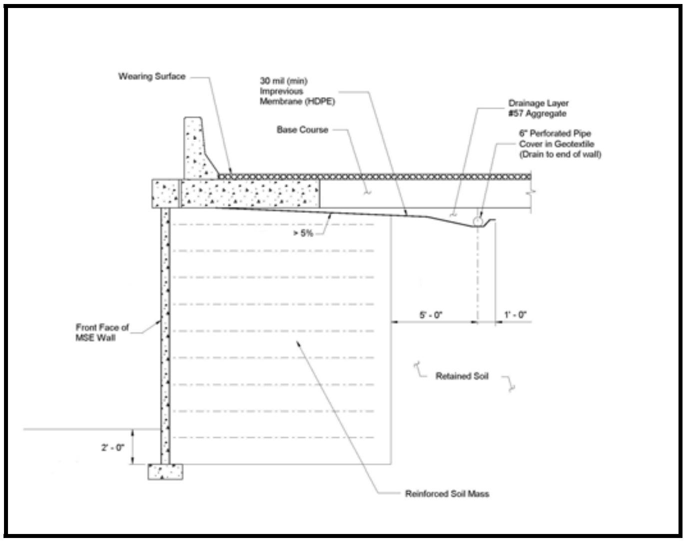


Figure C-17, Impervious Geomembrane Details (Earth Retaining Structures – June 2008)

C.10 SEISMIC DESIGN

The seismic external stability design shall conform to the requirements of Chapters 13 and 14. The seismic internal stability calculations shall conform to the requirements contained in the the AASHTO LRFD Bridge Design Specifications (Section 11.10 – Mechanically Stabilized Earth Walls), except all accelerations used shall conform to the requirements of this Manual. Additionally, all load and resistance factors shall conform to Chapters 8 and 9 and all displacements shall conform to Chapter 10.

C.11 COMPUTER SOFTWARE

A complete set of the MSE wall system supplier's design calculations prepared in accordance with this Appendix shall be provided by the MSE wall system supplier. The determination of all loading conditions and assumptions shall be fully documented with all design calculations. Submitted calculations (including computer runs) shall include all load cases that exist during construction and at the end of construction for any surcharges, hydraulic conditions, live loads, combinations, and obstructions within the reinforced backfill. Computer generated designs made by software other than FHWA's MSEW computer program shall meet the requirements of Chapter 26 and shall require verification that the computer program's design methodology meets the requirements provided herein. This shall be accomplished by either:

1. Complete, legible, calculations that show the design procedure step-by-step for the most critical geometry and loading condition that will govern each design section of the MSE wall structure. Calculations may be computer generated provided that all input, equations, and assumptions used are shown clearly.

2. A diskette with the input files and the full computer output of the FHWA sponsored computer program MSEW (latest version) for the governing loading condition for each design section of the MSE wall structure. This software may be obtained at:

ADAMA Engineering, Inc.
33 The Horseshoe, Covered Bridge Farms
Newark, Delaware 19711, USA
Tel. (302) 368-3197, Fax (302) 731-1001

C.12 REFERENCES

Elias, V., Christopher, B. R., and Berg, R. R. (2001), Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction, FHWA Publication No. FHWA-NHI-00-043, Department of Transportation, Federal Highway Administration, Washington D.C.

Elias, V., Salman, I., Juran, E., Pearce, E., and Lu, S. (1997), *Testing Protocols for Oxidation and Hydrolysis of Geosynthetics*, FHWA RD-97-144, Department of Transportation, Federal Highway Administration, Washington D.C.

Tanyu, B. F., Sabatini, P. J., and Berg, R. R. (2008), Earth Retaining Structures, FHWA-NHI-07-071, Department of Transportation, Federal Highway Administration, Washington D.C.

Appendix D

REINFORCED SOIL SLOPES

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

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APPENDIX D

REINFORCED SOIL SLOPE

DESIGN GUIDELINES

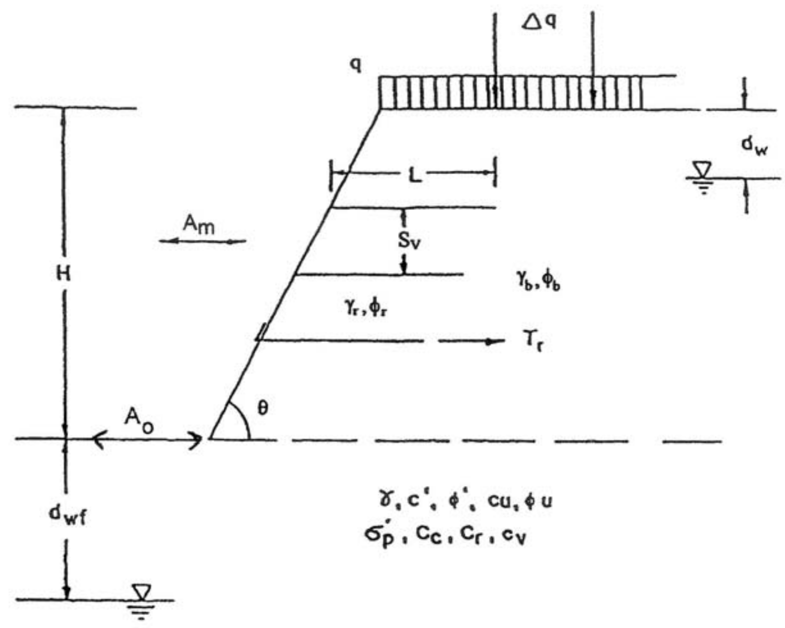
D.1 INTRODUCTION

This document outlines SCDOT's design methodology for Reinforced Soil Slopes (RSS). RSS structures are internally stabilized fill slopes, constructed of alternating layers of compacted soil and reinforcement. An RSS is different from an MSE wall or a conventional (unreinforced) slope in that the slope has an inclination ranging from 2H:1V to 1H:1V. This Appendix governs the design of permanent and temporary RSS structures. The design life of both permanent and temporary RSS is provided in Chapter 18.

This design process assumes that the existing subgrade soils provide a stable foundation for the founding the RSS. If improvement is required, see Chapters 19 and 20. This procedure assumes the classical, rotational, limit equilibrium slope stability methods are applicable (see Chapter 17). A circular arc failure surface is assumed in the design procedure of the RSS.

D.2 DESIGN CONSIDERATIONS AND REQUIREMENTS

The first part of the design is determining the geometry, the external loading conditions, the performance criteria and any construction constraints. The geometry should include the location relative to the remainder of the project (i.e. to the centerline and specific station). The geometry should also indicate the anticipated toe and crest of the slope (see Figure D-1). During this step of the design, external loads should be identified. These loads include, but are not limited to transient (traffic), permanent (weight of pavement surface) and/or seismically induced loads. The performance criteria are based on the Operational Classification of the Bridge or Roadway (see Chapter 8). The load and resistance factors are determined from Chapters 8 and 9, respectively. The performance limits are provided in Chapter 10. Any constraints on construction (i.e., soft ground, standing water, limited ROW, etc.) should also be identified during this step. These construction constraints should be carefully considered before deciding to use an RSS.



Notations:

H = slope height
θ = slope angle
T_r = strength of reinforcement
L = length of reinforcement
S_v = vertical spacing of reinforcement
q = surcharge load, q
Δq = temporary live load, Δq
A_m = design seismic acceleration
d_w = depth to the ground water table in slope
d_{wf} = depth to the ground water table in foundation
c_u and ϕ_u, or c' and ϕ' = strength parameters for each soil layer.
γ_{wet} and γ_{dry} = unit weights for each soil layer
C_c, C_r, c_v and σ'_p = consolidation parameters for each soil layer
A_O = ground acceleration coefficient
g = acceleration due to gravity

Figure D-1, RSS Design Requirements and Geometry
(MSEW and RSS D&C – March 2001)

D.3 SITE CONDITIONS

The second step in the design of RSS is the evaluation of the topography, subsurface conditions, and in-situ soil/rock parameters. The topography evaluation should include reviewing the height requirements of the slope, the amount of space between the toe of the slope and the anticipated extent of the reinforcement and the condition of the existing ground surface. This evaluation should identify the need for any temporary shoring that may be required to install the RSS (i.e. the grading of the site requires cutting). The subsurface conditions and in-situ soil/rock parameters shall be evaluated using the procedures presented in Chapters 4 through 7.

D.4 REINFORCED FILL MATERIAL PROPERTIES

The fill materials to be used to construct the RSS shall meet the criteria provided in the following tables. The actual soil strength parameters (φ, c and γ_t) shall be determined in accordance with Chapter 6.

Table D-1, Maximum Reinforced Backfill Properties

Material Property	Granular Backfill
Internal Friction Angle	34°
Total Unit Weight (lbs./cubic foot)	120

Table D-2, Granular Backfill Gradation Requirements

Sieve Size	Percent Passing
¾ in	100
No. 4	20-100
No. 40	0-60
No. 100	0-30
No. 200	0-15
Plasticity Index	≤ 6
Liquid Limit	≤ 30
C_U[1]	≥ 4[2]
Organic Content	< 1%

[1] $C_U = D_{60}/D_{10}$

[2] Pullout testing required for C_U less than 4

Table D-3, Electrochemical Properties of Reinforced Backfill

Reinforcement Material	Property	Criteria
Metallic	Resistivity[1]	>3,000 ohm-cm
Metallic	Chlorides	<100 ppm
Metallic	Sulfates	<200 ppm
Metallic/Geosynthetic	pH	4.5 < pH < 9

[1] Chloride and sulfate testing are not required if the resistivity is greater than 5,000 ohm-cm

Table D-4, Temporary RSS Backfill Properties

Gradation		Plasticity Index	Liquid Limit	C_U	pH	Internal Friction Angle	Total Unit Weight (pcf)	Organic Content
¾ in	No. 200	≤ 15	≤ 30	≥ 4	3 < pH < 10	> 28°	> 115	<1%
100%	$\leq 30\%$							

D.5 DESIGN PARAMETERS FOR REINFORCEMENT

Portions of the following sections of this Appendix are adopted directly from <u>Earth Retaining Structures</u>, FHWA-NHI-07-071, June 2008 and <u>Mechanically Stabilized Earth Walls and</u>

Reinforced Soil Slopes Design & Construction (MSEW and RSS D&C), FHWA-NHI-00-043, March 2001 and are used with the permission of the US Department of Transportation, Federal Highway Administration. Italics have been added to reflect additions or modifications to the selected text and to supply references to this Manual. According to Earth Retaining Structures:

In this design step, the maximum factored tensile stress in each reinforcement layer (T_{max}) is compared to the nominal long-term reinforcement design strength as presented below:

$$\boldsymbol{T_{\max} \leq \varphi R_c T_{al}} \qquad \text{Equation D-1}$$

Where,

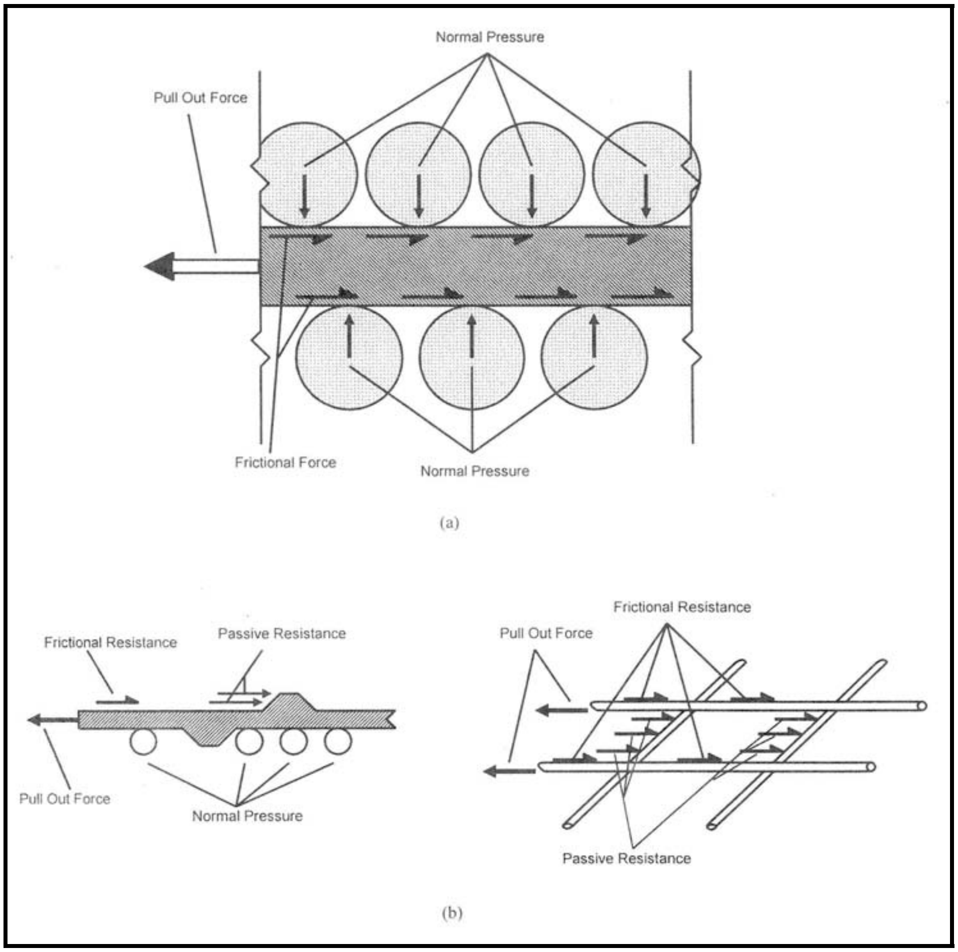
φ = Resistance factor for tensile rupture (*see Chapter 9*)
R_c = Reinforcement coverage ratio as defined in Equation *D-17 and Figure D-7*)
T_{al} =Nominal long-term reinforcement design strength

The nominal long-term reinforcement design strength (T_{al}) for LRFD is computed for inextensible and extensible reinforcements as presented *in the following sections.*

D.5.1 Reinforcement Pullout Resistance

"The purpose of this design step is to check the pullout resistance of the reinforcements. The resistance develops after the stress transfer between the soil and reinforcement takes place, which occurs through two mechanisms;

1. Friction along the soil-reinforcement interface (*see Figure D-2a)*
2. Passive soil resistance or lateral bearing capacity developed along the transverse sections of the reinforcement (*see Figure D-2b)*



(a) By friction (b) By passive resistance

Figure D-2, Mechanisms of Pullout Resistance
(Earth Retaining Structures – June 2008)

Stresses are transferred between soil and reinforcement by friction and/or passive resistance depending on reinforcement geometry.

Friction develops at locations where there is a relative shear displacement and corresponding shear stress between soil and reinforcement surface. Reinforcing elements where friction is important should be aligned with the direction of soil reinforcement relative movement. Examples of such reinforcing elements are steel strips, longitudinal bars in grids, geotextile, and some geogrid layers.

Passive Resistance occurs through the development of bearing type stresses on "transverse" reinforcement surfaces normal to the direction of soil reinforcement relative movement. Passive resistance is generally considered to be the primary interaction for rigid geogrids, bar mat, and wire mesh reinforcements. The transverse ridges on "ribbed" strip reinforcements also provide some passive resistance.

The contribution of each transfer mechanism for a particular reinforcement will depend on the roughness of the surface (skin friction), normal effective stress, grid opening dimensions, thickness of the transverse members, and elongation characteristics of the reinforcement. Equally important for interaction development are the soil characteristics, including grain size and grain size distribution, particle shape, density, water content, cohesion, and stiffness.

The primary function of reinforcement is to restrain soil deformations. In doing so, stresses are transferred from the soil to the reinforcement. These stresses are carried by the reinforcement in two ways: in tension or in shear and bending.

The unfactored pullout resistance (P_r) of the reinforcement per unit width of reinforcement is estimated as:

$$P_r = F^* \alpha \sigma_v \qquad \text{Equation D-2}$$

Parameters in Equation *D-2* are defined in Equation *D-16*.

In this design step, the reinforcement pullout resistance is evaluated at each reinforcement level. The required total length for reinforcement to generate appropriate pullout resistance for each level is calculated and then, compared against the total reinforcement length initially estimated *previously*. The initially estimated total reinforcement length may have to be adjusted based on the required length calculated in this step.

D.5.1.1 T_{al} for Inextensible Reinforcements

The nominal long-term design strength of inextensible reinforcement is provided below:

$$T_{al} = \frac{A_c F_y}{b} \qquad \text{Equation D-3}$$

Where,

F_y = Minimum yield strength of steel
b = Unit width of sheet, grid, bar or mat
A_c = Design cross sectional area corrected for corrosion loss

The lower resistance factor of 0.65 (*see Chapter 9*) for grid reinforcement (as compared to a resistance factor of 0.75 (*see Chapter 9*) for strip reinforcement) accounts for the greater potential for local overstress due to load nonuniformities for steel grids than for steel strips or bars.

A_c for strips is determined as:

$$A_c = bt_c = b(t_n - t_s) \qquad \text{Equation D-4}$$

Where,

b = Unit width of sheet, grid, bar or mat
t_c = Thickness at end of design life (see Figure *D-3*)
t_n = Thickness at end of construction
t_s = Sacrificial thickness of metal expected to be lost by uniform corrosion during the service life of the structure

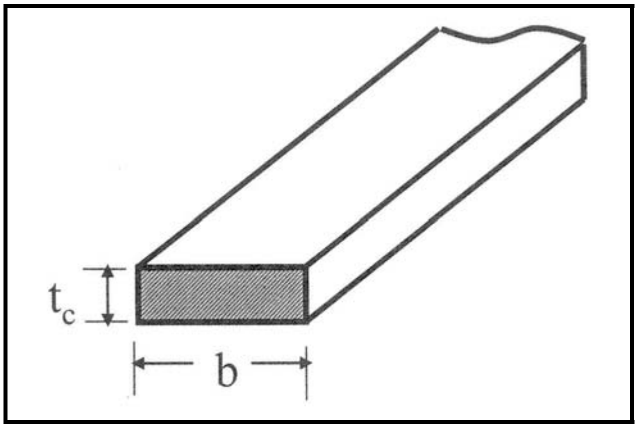


Figure D-3, Cross Section Area for Strips
(Earth Retaining Structures – June 2008)

When estimating t_s, it may be assumed that equal loss occurs from the top and bottom of the strip.

A_c for bars is determined as:

$$A_c = N_b\left(\frac{\pi D^{*2}}{4}\right) \qquad \text{Equation D-5}$$

Where,

N_b = Number of bars per unit width b
D* = Bar diameter after corrosion loss (Figure *D-4*)

When estimating D*, it may be assumed that corrosion losses occur uniformly over the area of the bar.

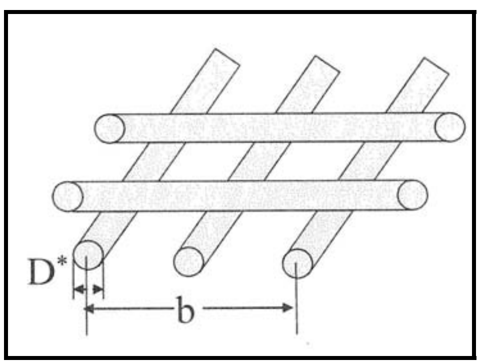


Figure D-4, Cross Section Area for Bars
(Earth Retaining Structures – June 2008)

D.5.1.2 Corrosion Rates

The corrosion rates presented below are suitable for conservative design. These rates assume a mildly corrosive backfill material having the controlled electrochemical property limits that are discussed *previously*.

Corrosion Rates – mildly corrosive backfill

For corrosion of galvanization on each side

- 0.58 mil./year/side (first 2 years)
- 0.16 mil./year/side (thereafter)

For corrosion of residual carbon steel on each side

- 0.47 mil./year/side (after zinc depletion)

Based on these rates, complete corrosion of galvanization with the minimum required thickness of 3.4 millimeters (*mil.*) (AASHTO M 111) is estimated to occur during the first 16 years and a carbon steel thickness or diameter loss of 0.055 inches to 0.08 inches would be anticipated over the remaining 75- to 100-year design life, respectively. The designer of an RSS structure should also consider the potential for changes in the reinforced backfill environment during the structure's service life. In certain parts of *South Carolina*, it can be expected that deicing salts might cause such an environment change. For this problem, the depth of chloride infiltration and concentration are of concern.

For permanent structures directly supporting roadways exposed to deicing salts, limited data indicate that the upper 8 feet of the reinforced backfill (as measured from the roadway surface) are affected by higher corrosion rates not presently defined. Under these conditions, it is recommended that a 30 mil (minimum) geomembrane be placed below the road base and tied into a drainage system to mitigate the penetration of the deicing salts in lieu of higher corrosion rates.

D.5.1.3 T_{al} for Extensible Reinforcements

The nominal long-term design strength of extensible reinforcement is provided below:

$$T_{al} = \frac{T_{ult}}{RF} \qquad \text{Equation D-6}$$

Where,

T_{ult} = Minimum average roll value ultimate tensile strength

RF = Combined strength reduction factor to account for potential long-term degradation due to installation damage, creep and chemical aging

$$RF = RF_{ID} RF_{CR} RF_{D} \qquad \text{Equation D-7}$$

Where,

RF_{ID} = Strength reduction factor to account for installation damage to reinforcement

RF_{CR} = Strength reduction factor to prevent long-term creep rupture of reinforcement

RF_{D} = Strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation

According to AASHTO (2007) values of RF_{ID}, RF_{CR}, RF_D shall be determined from product specific test results.

D.5.1.4 Reduction Factor RF_{ID}

The following sections of this Appendix are adopted directly from Earth Retaining Structures, FHWA-NHI-07-071, June 2008 and Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction (MSEW and RSS D&C), FHWA-NHI-00-043, March 2001 and are used with the permission of the US Department of Transportation, Federal Highway Administration. Italics have been added to reflect additions or modifications to the selected text and to supply references to this Manual.

> RF_{ID} can range from 1.05 to 3.0 depending on backfill gradation and product mass per unit weight. Even with product specific test results, the minimum reduction factor shall be 1.1 to account for testing uncertainties. The placement and compaction of the backfill material against the geosynthetic reinforcement may reduce its tensile strength. The level of damage for each geosynthetic reinforcement is variable and is a function of the weight and type of the construction equipment and the type of geosynthetic material. The installation damage is also influenced by the lift thickness and type of soil present on either side of the reinforcement. Where granular and angular soils are used for backfill, the damage is more severe than where softer, finer, soils are used. For a more detailed explanation of the RF_{ID} factor, see Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, FHWA-NHI-00-044.

D.5.1.5 Reduction Factor RF_{CR}

> RF_{CR} is obtained from long-term laboratory creep testing as detailed in Elias et al. (2001). This reduction factor is required to limit the load in the reinforcement to a level known as the creep limit, that will preclude creep rupture over the life of the structure. Creep in itself does not degrade the strength of the polymer. Creep testing is essentially a constant load test on multiple product samples, loaded to various percentages of the ultimate product load, for periods of up to 10,000 hours. The creep reduction factor is the ratio of the ultimate load to the extrapolated maximum sustainable load (i.e., creep limit) within the design life of the structure (e.g., several years for temporary structures, 75 to 100 years for permanent structures). Typical reduction factors as a function of polymer type are indicated in Table *D-5*.

Table D-5, Creep Reduction Factors
(Earth Retaining Structures – June 2008)

Polymer Type	RF_{CR}
Polyester	1.6 to 2.5
Polypropylene	4.0 to 5.0
Polyethylene	2.6 to 5.0

If no product specific creep reduction factors are provided, then the maximum creep reduction factor for a specific polymer shall be used. If the polymer is unknown, then an RF_{CR} of 5.0 shall be used.

D.5.1.6 Reduction Factor RF_D

RF_D is dependent on the susceptibility of the geosynthetic to be attacked by microorganisms, chemicals, thermal oxidation, hydrolysis and stress cracking and can vary typically from 1.1 to 2.0. Even with product specific tests results, the minimum reduction factor shall be 1.1. A protocol for testing to obtain this reduction factor has been described in Elias et al. (1997).

D.6 UNREINFORCED STABILITY

The overall (global) stability of the unreinforced slope is checked first to determine if reinforcement is required, if the potential for deep-seated failure surfaces is possible and to determine the approximate limit of reinforcement. If the resistance factor is less than required in Chapter 9, then, the unreinforced slope is stable and no reinforcement is required. (Note: The resistance factor (φ) is the inverse of the Factor of Safety (φ = 1/FS).) If the resistance factor is greater than indicated in Chapter 9, reinforcement of the slope is required. Further this stability check also identifies potential deep-seated failures. Deep-seated failure surfaces extend into the foundation soil and may require some form of ground improvement (see Chapters 19 and 20). Finally, this stability analysis is used to determine the first estimate of the length of reinforcement required to stabilize the slope.

This stability can be determined using classical slope stability analyses (see Chapter 17). The failure surfaces may be circular or non-circular (sliding block) and both should be checked. Overall stability analyses are performed for the Service limit state. The results of the overall stability analysis can, and does, effect the reinforcement length used in the design. It should be noted that it is assumed that all RSS are free draining and that pore water pressures are not allowed to build up behind the face of the slope.

After the development of the final design, a compound global stability analysis shall be performed. As defined in Chapter 18, a compound stability analysis examines failure surfaces that pass through either the retained fill and reinforced soil mass to exit through the RSS face or that pass through the retained fill, reinforced soil mass and the foundation soil to exit beyond the toe of the RSS. The actual strength parameters for the reinforced soil mass shall be used in the analysis. These analyses can only be performed once a specific reinforcement strength and type is selected.

D.7 REINFORCEMENT DESIGN

The reinforcement used in RSS may consist of either extensible (geosynthetics) or inextensible (metallic) reinforcement; however, inextensible reinforcement may only be used with wire baskets and must be connected to the baskets. In this step, the reinforcement is designed to provide a stable slope that meets the requirements of the project. According to MSEW and RSS D&C:

> Calculate the total reinforcement tension per unit width of slope T_S required to obtain the required *resistance factor* $1/\varphi_r$ for each potential failure surface inside the critical zone in *the previous step* that extends through or below the toe of the slope using the following equation:

$$T_S = \left(\frac{1}{\varphi_r} - \frac{1}{\varphi_u} \right) \frac{M_D}{D}$$ Equation D-8

Where,

T_S = The sum of the required tensile force per unit width of reinforcement (considering rupture and pullout) in all reinforcement layers intersecting the failure surface

M_D = Driving moment about the center of the failure surface

D = The moment arm of T_S about the center of the failure circle

$1/\varphi_r$ = Target minimum slope *resistance factor* which is applied to both the soil and reinforcement

$1/\varphi_u$ = Unreinforced slope *resistance factor*

The largest T_S calculated establishes the maximum total design tension ($T_{S\text{-max}}$) required. Determine the total design tension per unit width of slope ($T_{S\text{-MAX}}$) using Figure D-5. Compare $T_{S\text{-MAX}}$ from the chart to $T_{S\text{-max}}$ calculated from Equation D-8, if the results are significantly different, check the assumptions listed on the figure and check the calculations in the previous step (Unreinforced Stability) to determine if there is a larger potential failure surface not originally included in the analysis.

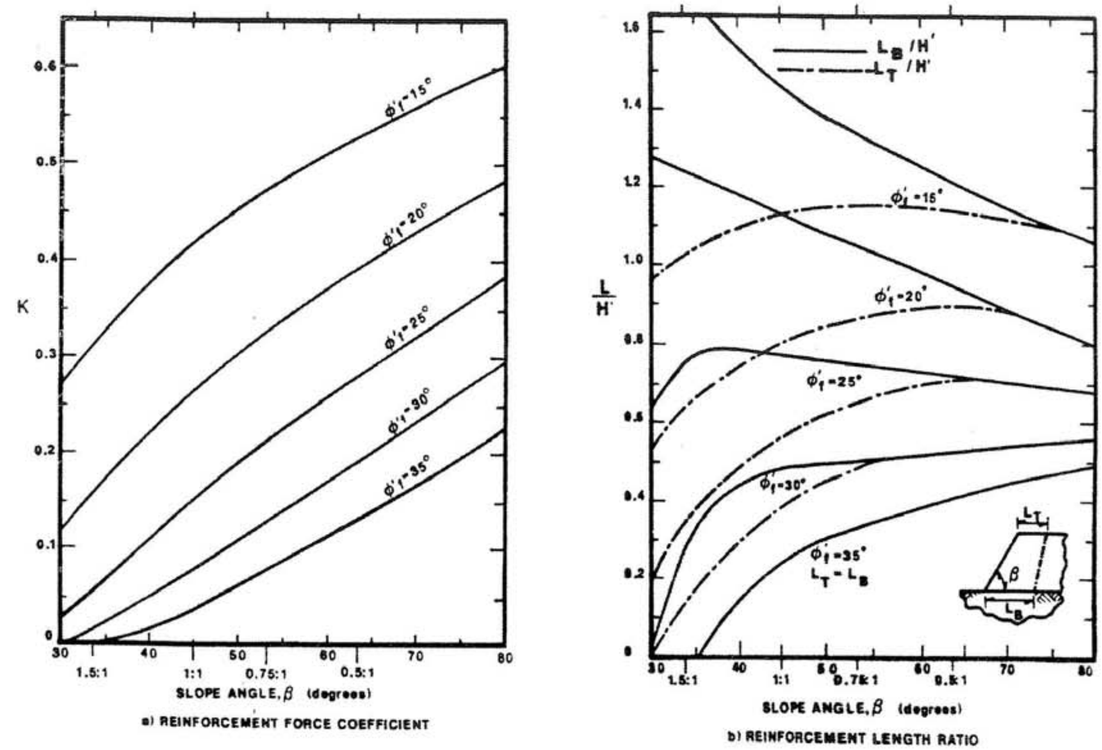


CHART PROCEDURE:

1) Determine force coefficient K from figure above, where ϕ_r = friction angle of reinforced fill:

$$\phi_f = \tan^{-1}\left(\frac{\tan\phi_r}{FS_R}\right)$$

2) Determine:

$$T_{S\text{-}MAX} = 0.5\ K\ \gamma_r\ (H')^2$$

where: $H' = H + q/\gamma_r$
q = a uniform load

3) Determine the required reinforcement length at the top L_T and bottom L_B of the slope from the figure above.

LIMITING ASSUMPTIONS

- Extensible reinforcement.
- Slopes constructed with uniform, cohesionless soil, $c = 0$).
- No pore pressures within slope.
- Competent, level foundation soils.
- No seismic forces.
- Uniform surcharge not greater than $0.2\ \gamma_r\ H$.
- Relatively high soil/reinforcement interface friction angle, $\phi_{sg} = 0.9\ \phi_r$ (may not be appropriate for some geotextiles).

Note: $FS_R = 1/\varphi_R$, where φ_R is the resistance factor (see Chapter 9)

Figure D-5, Reinforcement Strength Requirements Chart Solution (MSEW and RSS D&C – March 2001)

According to MSEW and RSS D&C"

> Figure *D-5* is provided for a quick check of computer-generated results. The figure presents a simplified method based on a two-part wedge type failure surface and is limited by the assumptions noted on the figure.
>
> Note that Figure *D-5* is not intended to be a single design tool. Other design charts that are available from the literature, could also be used (*e.g. Werner and Resl, 1986; Ruegger, 1986; Leshchinsky and Boedeker, 1989; and Jewell, 1990*). Several computer programs are also available (*see Section D.12*) for analyzing a slope with a given reinforcement and can be used as a check. Judgment in selection of other appropriate design methods (i.e., most conservative or experience) is required.

After determining the maximum required tensile strength of the geotextile, the determination of the distribution of the reinforcement comes next. According to MSE and RSS D&C:

> For low slopes ($H \leq 20$ feet) assume a uniform reinforcement distribution and use $T_{S\text{-}max}$ to determine the spacing or the required tension, T_{max}, requirements for each reinforcement layer.
>
> For high slope ($H > 20$ feet), divide the slope into two (top and bottom) or three (top, middle and bottom) reinforcement zones of equal height and use a factored $T_{S\text{-}max}$ in each zone for spacing or design tension requirements. The total required tension in each zone is found from:
>
> For two zones:

$$T_{Bottom} = \tfrac{3}{4} T_{S-\max} \qquad \text{Equation D-9}$$

$$T_{Top} = \tfrac{1}{4} T_{S-\max} \qquad \text{Equation D-10}$$

> For three zones:

$$T_{Bottom} = \tfrac{1}{2} T_{S-\max} \qquad \text{Equation D-11}$$

$$T_{Middle} = \tfrac{1}{3} T_{S-\max} \qquad \text{Equation D-12}$$

$$T_{Top} = \tfrac{1}{6} T_{S-\max} \qquad \text{Equation D-13}$$

> The force ($T_{S\text{-}max}$) is assumed to be uniformly distributed over the entire zone.
>
> Determine reinforcement vertical spacing (S_V) or the maximum design tension (T_{max}) requirements for each reinforcement layer.

For each zone, calculate T_{max} for each reinforcing layer in that zone based on an assumed S_V, or, if the allowable reinforcement strength is known, calculate the minimum vertical spacing and number of reinforcing layers N required for each zone based on:

$$T_{\max} = \frac{T_{zone} S_V}{H_{zone}} = \frac{T_{zone}}{N} \leq T_a R_c \qquad \text{Equation D-14}$$

Where,

R_c = Coverage ratio of the reinforcement which equals the width of the reinforcement b divided by the horizontal spacing S_h

S_V = Vertical Spacing of reinforcement; multiples of compacted layer thickness of ease of construction

T_{zone} = Maximum reinforcement tension required for each zone; $T_{S\text{-}max}$ for low slopes (H $\leq$ 20 feet)

$T_a = T_{al}$

H_{zone} = Height of zone; T_{Top}, T_{Middle}, and T_{Bottom} for high slopes (H > 20 feet)

N = Number of reinforcement layers

Use short (*4 to 6 feet*) lengths of intermediate reinforcement layers to maintain a maximum vertical spacing of 16 inches or less for face stability and compaction quality. For slopes flatter than 1H:1V (*45°*), closer spaced reinforcements (i.e., every lift or every other lift, but no greater than 16 inches) preclude having to wrap the face in well graded soils (e.g., sandy gravel and silty and clayey sands). Wrapped faces are required for steeper slopes and uniformly graded soils to prevent face sloughing. Alternative vertical spacings could be used to prevent face sloughing, but in these cases a face stability analysis should be performed either using the method presented in this chapter or by evaluating the face as an infinite slope using:

$$\frac{1}{\varphi} = FS = \frac{c'H + (\gamma_g - \gamma_w) Hz \cos^2 \beta \tan\varphi' + F_g (\cos\beta \sin\beta + \sin^2 \tan\varphi')}{\gamma_g Hz \cos\beta \sin\beta} \qquad \text{Equation D-15}$$

Where,

c’ = Effective cohesion

φ’ = Effective friction angle

γ_g = Saturated unit weight

γ_w = Unit weight of water

z = Vertical depth to failure plane defined by the depth to saturation

H = Vertical slope height

β = Slope angle

F_g = Summation of geosynthetic resisting force

Intermediate reinforcement should be placed in continuous layers and does not need to be as strong as the primary reinforcement, but it must be strong enough to survive construction (e.g., minimum survivability requirements for geotextiles in road stabilization applications in AASHTO M-288) and provide localized tensile reinforcement to the surficial soils.

If the interface friction angle of the intermediate reinforcements, ρ_{sr}, is less than that of the primary reinforcement ρ_r, then ρ_{sr} should be used in the analysis for the portion of the failure surface intersecting the reinforced soil zone."

To ensure that the rule-of-thumb reinforcement distribution is adequate for critical or complex structures, recalculate T_S using *Equation D-8* to determine potential failure above each layer of primary reinforcement.

Check that the sum of the reinforcement forces passing through each failure surface is greater than T_S required for that surface. Only count reinforcement that extends *more than 3 feet* beyond the surface to account for pullout resistance. If the available reinforcement force is not sufficient, increase the length of reinforcement not passing through the surface or increase the strength of lower-level reinforcement. Simplify the layout by lengthening some reinforcement layers to create two or three sections of equal reinforcement length for ease of construction and inspection. Reinforcement layers do not generally need to extend to the limits of the critical zone, except for the lowest levels of each reinforcement section. Check the length using Figure *D-5(b)*. Note: L_e is already included in the total length, L_T and L_B from *Figure D-5(b).*

When checking a design that has zones of different reinforcement lengths, lower zones may be over reinforced to provide reduced lengths of upper reinforcement levels. In evaluating the length of requirements for such cases, the pullout stability for the reinforcement must be carefully checked in each zone for the critical surfaces exiting at the base of each length zone.

D.7.1.1 Estimating L_e

The length of reinforcement in the resisting zone (L_e) is determined using the following equation:

$$L_e \geq \frac{T_{max}}{\varphi F^* \alpha \sigma'_v 2CR_c} \qquad \text{Equation D-16}$$

Where,

T_{max} = Maximum factored tensile load in the reinforcement (calculated in *Equation D-8)*

φ = Resistance factor for reinforcement pullout (*see Chapter 9*)

F^* = Pullout friction factor (discussed below)

α = Scale effect correction factor (discussed below)

σ_v = Unfactored vertical stress at the reinforcement level in the resistance zone

C = Overall reinforcement surface area geometry factor (2 for strip, grid and sheet-type reinforcement)

R_c = Reinforcement coverage ratio (*see Figure D-7)*

$$R_c = \frac{b}{S_h}$$ Equation D-17

Where,

b = Gross width of the reinforcing element

S_h = Center-to-center horizontal spacing between reinforcements

D.7.1.2 Correction Factor (α)

The correction factor (α) depends primarily upon the strain softening of the compacted granular backfill material, the extensibility, and the length of the reinforcement. Typical values of α based on reinforcement type are presented in Table *D-6*. For inextensible reinforcement, α is approximately 1, but it can be substantially smaller than 1 for extensible reinforcements. The α factor can be obtained from pullout tests on reinforcements with different lengths or derived using analytical or numerical load transfer models, which have been "calibrated" through numerical test simulations. In the absence of test data, *the values included in Table D-6 should be used for geogrids and geotextiles.*

Table D-6, Typical Values of α
(Earth Retaining Structures – June 2008)

Reinforcement Type	α
All steel reinforcements	1.0
Geogrids	0.8
Geotextiles	0.6

D.7.1.3 Pullout Friction Factor (F*)

The pullout friction factor can be obtained most accurately from laboratory or field pullout tests performed with the specific material to be used on the project (i.e., select backfill and reinforcement). Alternatively, F* can be derived from empirical or theoretical relationships developed for each soil-reinforcement interaction mechanism and provided by the reinforcement supplier. For any reinforcement, F* can be estimated using the general equation:

$$F^* = F_q \alpha_\beta + \tan\rho$$ Equation D-18

Where,

F_q = The embedment (or surcharge) bearing capacity factor

α_β = A bearing factor for passive resistance which is based on the thickness per unit width of the bearing member

ρ = The soil-reinforcement interaction friction angle

Equation *D-18* represents systems that have both the frictional and passive resistance components of the pullout resistance. In certain systems, however, one component is much smaller than the other and can be neglected for practical purposes. In absence of site-specific pullout test data, it is reasonable to use these semi-empirical relationships in conjunction with the standard specifications for backfill to provide a conservative evaluation of pullout resistance.

For steel ribbed reinforcement, F* is commonly estimated as:

$$\boldsymbol{F^* = \tan\rho = 1.2 + \log C_u} \qquad \text{Equation D-19}$$

at the top of the structure =2.0 maximum

$$\boldsymbol{F^* = \tan\varphi_r} \qquad \text{Equation D-20}$$

at a depth of 20 feet and below

Where,

ρ = Interface friction angle mobilized along the reinforcement

φ_r = Reinforced backfill peak friction angle

C_u = Uniformity coefficient of the backfill (*see Chapter 6*)

If the specific C_u for the wall backfill is unknown during design, a C_u of 4 should be assumed (i.e., F* = 1.8 at the top of the wall), for backfill meeting the requirements *previously provided.*

For steel grid reinforcements with transverse spacing (S_t) $\geq$ 6 inches, F* is a function of a bearing or embedment factor (F_q), applied over the contributing bearing factor (α_β), as follows:

$$\boldsymbol{F^* = F_q\alpha_\beta = 40\alpha_\beta = 40\left(\frac{t}{2S_t}\right) = 20\left(\frac{t}{S_t}\right)} \qquad \text{Equation D-21}$$

at the top of the structure

$$\boldsymbol{F^* = F_q\alpha_\beta = 20\alpha_\beta = 20\left(\frac{t}{2S_t}\right) = 10\left(\frac{t}{S_t}\right)} \qquad \text{Equation D-22}$$

at a depth of 20 feet and below

Where,

t = The thickness of the transverse bar

S_t = *The distance between individual bars in steel grid reinforcement and* shall be uniform throughout the length of the reinforcement, rather than having transverse grid members concentrated only in the resistance zone

For geosynthetic (i.e., geogrid and geotextile) sheet reinforcement, the pullout resistance is based on a reduction in the available soil friction, with the reduction factor often referred to as an interaction factor (C_i). In the absence of test data, the F* value for geosynthetic reinforcement should conservatively be estimated as:

$$\boldsymbol{F^* = 0.67\tan\varphi_r} \qquad \text{Equation D-23}$$

The relationship between F* and depth below the top of the wall for different reinforcement types is summarized in Figure *D-6*.

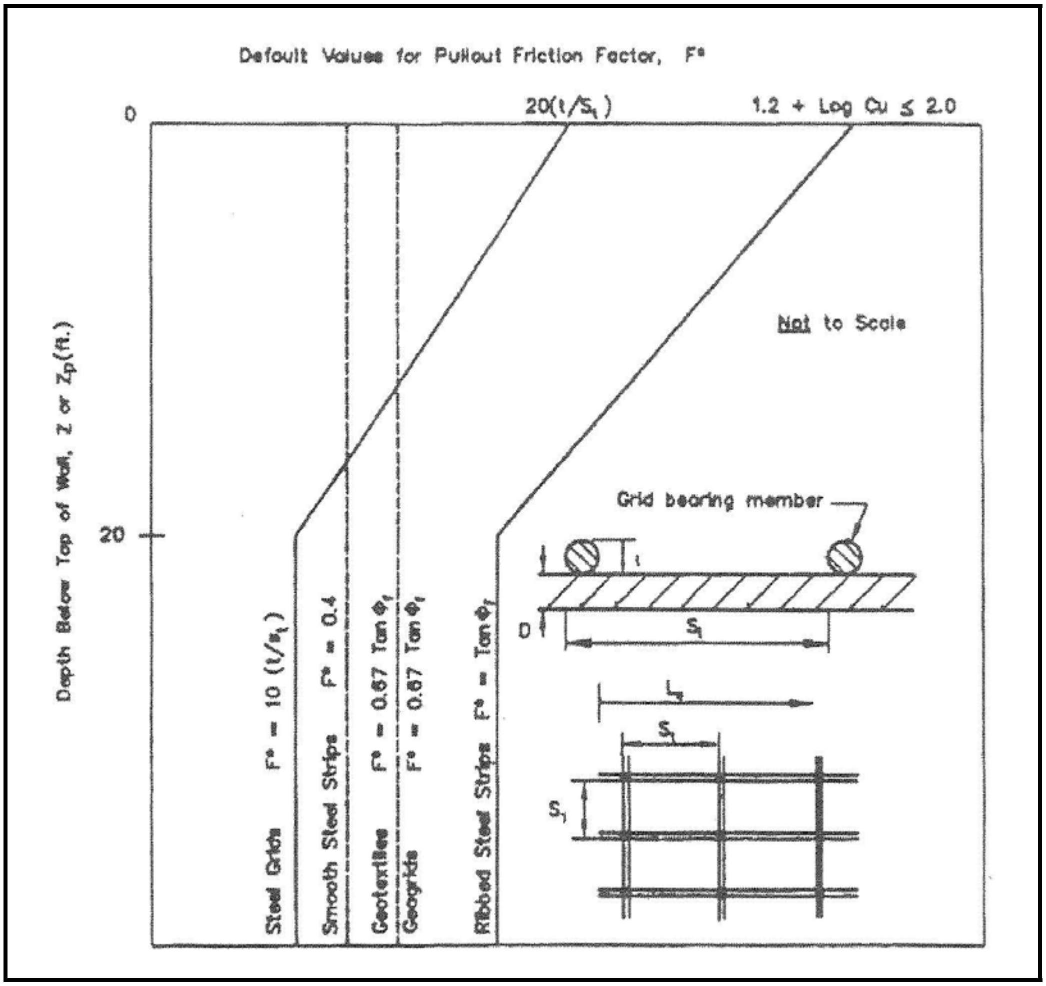
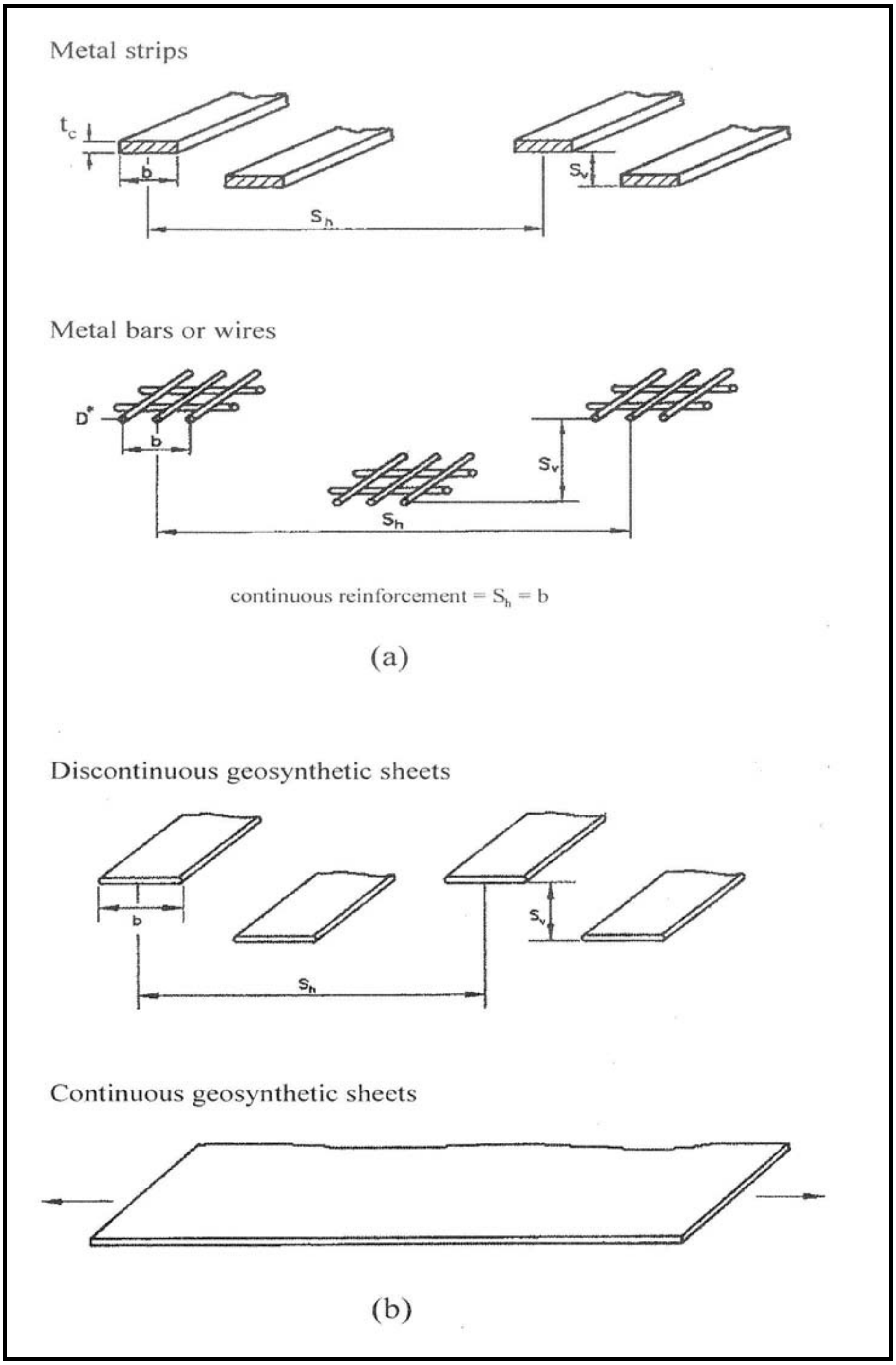


Figure D-6, Typical Values of F*
(Earth Retaining Structures – June 2008)



(a) Metal reinforcement (b) Geosynthetic reinforcement

Figure D-7, Definitions of b, S_h and S_V
(Earth Retaining Structures – June 2008)

D.8 SELECTION OF REINFORCEMENT

In this step, the type of reinforcement shall be determined. The two types of reinforcement are extensible and inextensible. Extensible reinforcements consist of geosynthetic materials, typically geogrids (biaxial or uniaxial) and geotextiles. These reinforcements are a wrapped face consisting of a layer of geogrid that wraps around the face and a layer of geotextile to prevent erosion of the reinforced soil materials. Inextensible reinforcements consist of bars or bar mats (metallic grids). These reinforcements are typically connected to wire baskets at the front face to provide anchorage at the face of the slope. The selection of the type of reinforcement is influenced by the strength required to maintain stability and the aesthetic appearance required at the completion of the project.

D.9 EXTERNAL STABILITY

D.9.1 Sliding Resistance

According to MSE and RSS D&C:

> Evaluate the width of the reinforced soil mass at any level to resist sliding along the reinforcement. A wedge type failure surface defined by the limits of the reinforcement (the length of reinforcement at the depth of evaluation defined *previously*). The analysis can best be performed using a computerized method which takes into account all soil strata and interface friction values. The back of the wedge should be angled at 45° +φ/2 (*see Figure D-8*) or parallel to the back of the reinforced zone, which ever is flatter (i.e., the wedge should not pass through layers of reinforcement to avoid an overly conservative design).

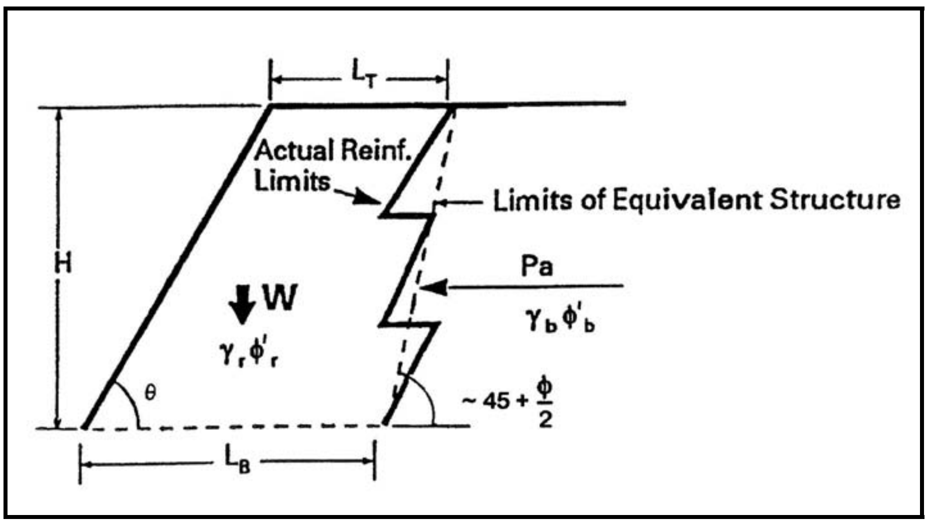


Figure D-8, Sliding Stability Analysis
(MSEW and RSS D&C – March 2001)

> A simple analysis using a sliding block method can be performed as a check. In this method, an active wedge is assumed at the back of the reinforced soil mass with the back of the wedge extending up at an angle of 45° +φ/2. Using this assumption, the driving force is equal to the active earth pressure and the resisting force is the frictional resistance provided by the weakest layer, either the reinforced soil, the foundation soil or the soil-reinforcement interface. The following relationships are then used:

Resistance Factor(Resisting Force) = Sliding Force Equation D-24

$$\varphi(\boldsymbol{W} + \boldsymbol{P_a}\ \mathbf{sin}\,\varphi_b)\,\mathbf{tan}\,\varphi_{\mathbf{min.}} = \boldsymbol{P_a}\ \mathbf{cos}\,\varphi_b \qquad \text{Equation D-25}$$

$$W = \frac{1}{2} L^2 \gamma_r \tan\theta \qquad \text{Equation D-26}$$

for L < H

$$W = \left[LH - \frac{H^2}{(2\tan\theta)} \right] \gamma_r \qquad \text{Equation D-27}$$

for L > H

$$P_a = \frac{1}{2} \gamma_b H^2 K_a \qquad \text{Equation D-28}$$

Where,

L = Length of bottom reinforcing layer in each level where there is a reinforcement length change

H = Height of Slope

φ = Resistance Factor (see Chapter 9)

$\varphi_{min,}$ = Minimum angle of shearing friction either between reinforced soil and reinforcement or the friction angle of the foundation soil

θ = Slope angle

γ_r & γ_b = Unit weight of the reinforced backfill and retained backfill, respectively

φ_b = Friction angle of retained fill (Note: If drains/filters are placed on the backslope, then φ_b equals the interface friction angle between the geosynthetic and retained fill)

D.9.2 Global (Deep-Seated) Stability

This sub-step is to evaluate the potential for deep-seated failure surfaces beyond or below the reinforced soil mass to provide resistance factors that meet the requirements of Chapter 9. This check is similar to and may use the results of the Unreinforced Stability analysis discussed previously.

D.9.3 Local Bearing Failure at Toe

According to MSE and RSS D&C:

If a weak layer exists beneath the embankment to limited depth D_S, which is less than the width of the slope b' (*see Figure D-9*), the *resistance factor* against failure by squeezing may be calculated from:

$$\varphi_{squeezing} = \frac{\gamma D_S \tan\theta}{2c_u} + \frac{H\gamma}{4.14c_u} \leq 0.75 \qquad \text{Equation D-29}$$

Where,

θ = Angle of slope

γ = Unit weight of soil in slope
D_S = Depth of soft soil beneath slope base of the embankment
H = Height of slope
c_u = Undrained shear strength of soft soil beneath slope

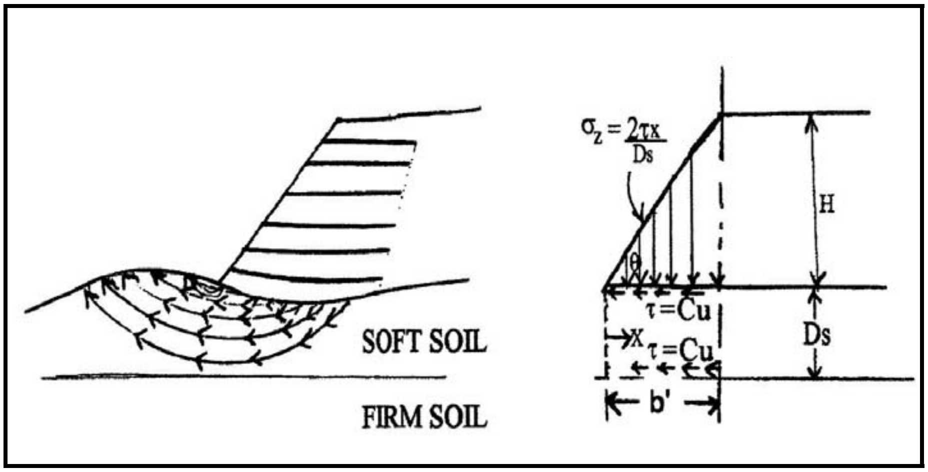


Figure D-9, Local Bearing Failure (Lateral Squeeze)
(MSEW and RSS D&C – March 2001)

Caution is advised and rigorous analysis (i.e., numerical modeling) should be performed when the *resistance factor (φ) is greater than 0.5*. This approach is somewhat conservative as it does not provide any influence from the reinforcement. When the depth of the soft layer, D_S, is greater than the base width of the slope, b', general slope stability will govern design.

D.9.4 Foundation Settlement

The settlement (total, differential and time rate) of the RSS shall be determined using the procedures provided in Chapter 17.

D.10 WALL DRAINAGE SYSTEM DESIGN

The following section of this Appendix is adopted directly from Earth Retaining Structures, FHWA-NHI-07-071, June 2008 and Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction (MSEW and RSS D&C), FHWA-NHI-00-043, March 2001 and are used with the permission of the US Department of Transportation, Federal Highway Administration. Italics have been added to reflect additions or modifications to the selected text and to supply references to this Manual.

D.10.1 Subsurface Water Control

Design of subsurface water drainage features should address flow rate, filtration, placement, and other details. Drains are typically placed at the rear of the reinforced soil mass in Figure *D-10*. Geocomposite drainage systems or conventional granular blanket and trench drains could be used. Granular drainage systems are not addressed in this *Appendix*.

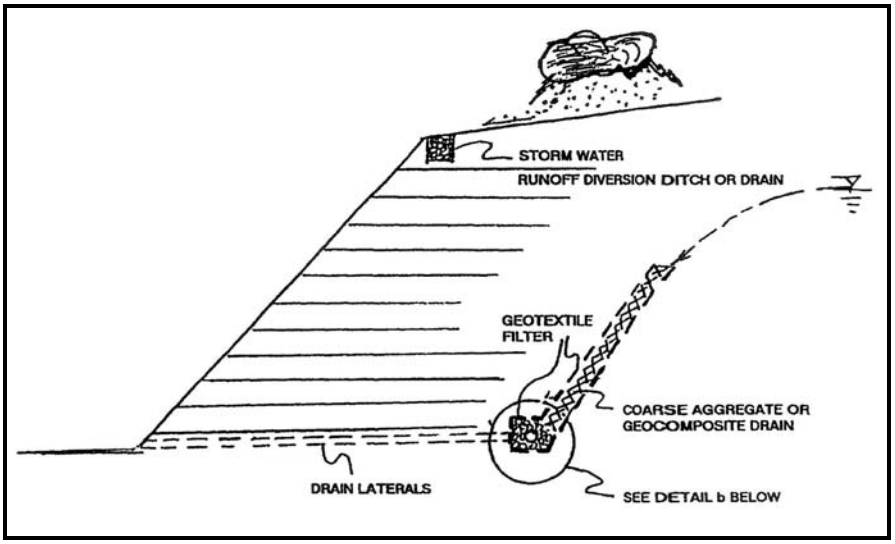


Figure D-10, Groundwater and Surface Drainage
(MSEW and RSS D&C – March 2001)

Lateral spacing of outlets is dictated by site geometry and estimated flow. Outlet design should address long-term performance and maintenance requirements. Geosynthetic drainage composites can be used in subsurface water drainage design. Drainage composites should be designed with consideration for:

1. Geotextile filtration/clogging
2. Long-term compressive strength of polymeric core
3. Reduction of flow capacity due to intrusion of geotextile into the core
4. Long-term inflow/outflow capacity

Procedures for checking geotextile permeability and filtration/clogging criteria are presented in Geosynthetic Design and Construction Guidelines, *August 2008, FHWA NHI-07-092*. Long-term compressive stress and eccentric loadings on the core of a geocomposite should be considered during design and selection. Intrusion of the geotextiles into the core and long-term outflow capacity should be measured with a sustained transmissivity test. Slope stability analyses should account for interface shear strength along a geocomposite drain. The geocomposite/soil interface will most likely have a friction value that is lower than that of the soil. Thus, a potential failure surface may be induced along the interface. Geotextile reinforcements (primary and intermediate layers) must be more permeable than the reinforced fill material to prevent a hydraulic build up above the geotextile layers during precipitation. Special emphasis on the design and construction of subsurface drainage features is recommended for structures where drainage is critical for maintaining slope stability. Redundancy in the drainage system is also recommended for these cases.

D.10.2 Surface Water Runoff

Surface water runoff should be collected above the reinforced slope and channeled or piped below the based of the slope. Wrapped faces and/or intermediate layers of secondary reinforcement may be required at the face of reinforced slopes to prevent local sloughing. Intermediate layers of reinforcement help achieve compaction at the face, thus increasing soil shear strength and erosion resistance. These layers also act as reinforcement against shallow or sloughing types of slope failures. Intermediate reinforcement is typically placed on each or every other soil lift, except at lifts where primary structural reinforcement is placed. Intermediate reinforcement also is placed horizontally, adjacent to primary reinforcement, and at the same elevation as the primary reinforcement when primary reinforcement is placed at less than 100 percent coverage in plan view. The intermediate reinforcement should extend 4 to 7 feet into the fill from the face. Select a long-term facing system to prevent or minimize erosion due to rainfall and runoff on the face.

Calculated flow-induced tractive shear stress on the face of the reinforced slope by:

$$\lambda = \boldsymbol{d}\gamma_{\boldsymbol{w}}\boldsymbol{s} \qquad \text{Equation D-30}$$

Where,

λ = Tractive shear stress
d = Depth of water flow
γ_w = Unit weight of water
s = The vertical to horizontal angle of slope face

For λ < *0.0145 pound per square inch (psi)*, consider vegetation with temporary or permanent erosion control mat. For λ > *0.0145 psi*, consider vegetation with permanent erosion control mat or other armor type systems (e.g., riprap, gunite, prefabricated modular units, fabric-formed concrete, etc.). Select vegetation based on local horticultural and agronomic considerations and maintenance. Select synthetic (permanent) erosion control mat that is stabilized against ultra-violet light and is inert to naturally occurring soil-born chemicals and bacteria. Erosion control mats and blankets vary widely in type, cost, and more importantly, applicability to project conditions. Slope protection should not be left to the construction contractor or vendor's discretion.

D.11 SEISMIC DESIGN

The seismic stability design shall conform to the requirements of Chapters 13 and 14. All load and resistance factors shall conform to Chapters 8 and 9. In addition, all displacements shall conform to Chapter 10.

D.12 COMPUTER SOFTWARE

The following section of this Appendix is adopted directly from Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction (MSEW and RSS D&C), FHWA-NHI-00-043, March 2001 and are used with the permission of the US Department of Transportation, Federal Highway Administration. Italics have been added to reflect additions or modifications to the selected text and to supply references to this Manual.

An alternative to reinforcement design is to develop a trial layout of reinforcement and analyze the reinforced slope with a computer program. Layout includes number, length, design strength, and vertical distribution of the geosynthetic reinforcement. The charts presented in Figure *D-5* provide a method for generating a preliminary layout. Note that these charts were developed with the specific assumptions noted in this figure.

Analyze the reinforced soil slope with the trial geosynthetic reinforcement layouts. The most economical reinforcement layout must provide the *maximum stability resistance factors* for internal, external and compound failure planes. A contour plot of the *highest resistance factor* values about the trial failure circle centroids is recommended to map and locate the *maximum resistance factor* values for the three modes of failure.

Computer generated designs made by software other than FHWA's ReSSA computer program shall meet the requirements of Chapter 26 and shall require verification that the computer program's design methodology meets the requirements provided herein. This shall be accomplished by either:

1. Provide complete, legible, calculations that show the design procedure step-by-step for the most critical geometry and loading condition that will govern each design section of the RSS structure. Calculations may be computer generated provided that all input, equations, and assumptions used are shown clearly.

2. Provide a diskette with the input files and the full computer output of the FHWA sponsored computer program ReSSA (latest version) for the governing loading condition for each design section of the RSS structure. This software may be obtained at:

ADAMA Engineering, Inc.
33 The Horseshoe, Covered Bridge Farms
Newark, Delaware 19711, USA
Tel. (302) 368-3197, Fax (302) 731-1001

D.13 REFERENCES

Elias, V., Christopher, B. R., and Berg, R. R. (2001), *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction*, FHWA Publication No. FHWA-NHI-00-043, Department of Transportation, Federal Highway Administration, Washington D.C.

Elias, V., Salman, I., Juran, E., Pearce, E., and Lu, S. (1997), *Testing Protocols for Oxidation and Hydrolysis of Geosynthetics*, FHWA RD-97-144, Department of Transportation, Federal Highway Administration, Washington D.C.

Jewell, R. A., (1990), "Revised Design Charts for Steep Reinforced Slopes, Reinforced Embankments: Theory and Practice in the British Isles", *Thomas Telford*, London, United Kingdom.

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Ruegger, R., (1986), "Geotextile Reinforced Soil Structures on which Vegetation can be Established", *Proceedings of the 3rd International Conference on Geotextiles*, Vienna, Austria, Volume II.

Tanyu, B. F., Sabatini, P. J., and Berg, R. R. (2008), *Earth Retaining Structures*, FHWA-NHI-07-071, Department of Transportation, Federal Highway Administration, Washington D.C.

Werner, G. and Resl, S., (1986), “Stability Mechanisms in Geotextile Reinforced Earth-Structures”, *Proceedings of the 3rd International Conference on Geotextiles*, Vienna, Austria, Volume II.

Appendix E

GEOTECHNICAL TEMPLATE PLANS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

APPENDIX E

GEOTECHNICAL TEMPLATE PLANS

The following list contains the currently available Geotechnical Template Plans. Consultants can obtain any of these plans via the SCDOT internet website at *Bridge Drawings and Details*.

Geotechnical General Notes
Ground Modification (Vibro-Stone Columns)
Earthquake Drains
Axial Shaft Load Test – XX" Dia. – Statnamic
Lateral Shaft Load Test – XX" Dia. – Statnamic
Axial Shaft Load Test – XX" Dia. – Osterberg Cell
Prefabricated Vertical Drains with Fabric

Appendix F

Project Specific Specifications List

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

APPENDIX F

PROJECT SPECIFIC SPECIFICATIONS LIST

The following list contains the currently available Project Specific Specifications. Consultants can obtain any of these specifications by contacting the PCS/GDS. Any changes made to the specifications, regardless of whether the change is made by a consultant or by a GDS, shall be highlighted prior to the review process to facilitate the review.

MSE Walls
Geogrid Soil Reinforcement
Ground Modification (Vibro Replacement)
Lightweight Aggregates
Pile Load Test
Shaft Load Test
Magnetic Extensometers
Monitoring Devices – Piezometers
Monitoring Devices – Slope Indicator
Prefabricated Earthquake Drain with Filter Fabric
Prefabricated Vertical Drain with Fabric
Reinforced Soil Slopes
Settlement Plates
Settlement Sensors
Vibration Monitoring

Appendix G

SCDOT Software List

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

APPENDIX G

SCDOT SOFTWARE LIST

The following list contains the software (both commercially available and non-commerical) used by SCDOT. Consultants are not required to have the same software as the Department. This is provided for reference to the consultants. The non-commercial software is used exclusively internal to the Department.

Commercial Software

Slope/ERS Software
GSTABL7 with STEDwin
STEDwin
PYWall
MSEW
ReSSA
EMBANK
FoSSA

Foundation Software
Driven
SPT97
LPILE
Shaft
SPILE
FB-Pier
GRLWEAP

Seismic Software
DEEPSOIL
SHAKE2000
DMOD
SeismoSignal
Newmark Deformation
LiquefyPro

Non-Commercial Software

Seismic Software
SCENARIO-PC
SCDOT SHAKE
ADRS Three-Point Method (excel spreadsheet)

Foundation Software
Point-of-Fixity (excel spreadsheet – used in preliminary design (only))