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A web-based information system for geoconstruction technologies and performance of stone column reinforced ground

Samuel Caleb Douglas
Iowa State University

A web-based information system for geoconstruction technologies and performance of stone column reinforced ground

by

Samuel Caleb Douglas

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Program of Study Committee:
Vernon R. Schaefer, Major Professor
Jeramy C. Ashlock
Neal R. Iverson
Kelly C. Strong
David J. White

Iowa State University

Ames, Iowa

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ABSTRACT

Geoconstruction technologies provide solutions for pavements, foundations, slopes, and retaining walls on transportation projects across the United States. This dissertation includes a selection of papers to address two knowledge gaps within the broad application area of geoconstruction technologies. First, the geotechnical engineering community lacks a central repository that summarizes, distills, and distributes the abundant information regarding geoconstruction technologies. Second, the stone column geoconstruction technology lacks design guidance for estimating settlements and the development of future project considerations based on a review of case histories.

The first two papers, "Web-Based Information System for Geoconstruction Technologies in Transportation Infrastructure" and "Selection Assistance for the Evaluation of Geoconstruction Technologies," describe a new information system that compiles the critical knowledge for 46 geoconstruction technologies applicable to transportation infrastructure from the following areas: ground improvement, geosynthetics, grouting, slope stabilization, soil reinforcement, soil stabilization, alternative materials, and recycling. The information system contains an introduction to the Geotechnical Design Process, Catalog of Technologies, Technology Selection Assistance, and Glossary. For each technology, the following documents can be accessed through the Catalog of Technologies: *Technology Fact Sheet*, *Photographs*, *Case Histories*, *Design Guidance*, *Quality Control/ Quality Assurance*, *Cost Information*, *Specifications*, and *Bibliography*. Technology selection assistance aids the user in identifying potential geoconstruction technologies for a user-defined set of project conditions.

The last two papers, “Reliability of Estimating Settlements for Stone Columns” and “Stone Columns: Lessons Learned, Settlements, and Future Project Considerations,” relate to the performance of ground reinforced by stone columns. The design considerations for stone columns were developed from a thorough review of published literature focusing on case histories that document both the successful and unsuccessful implementation of stone columns. Over 15 methods of estimating settlements were identified for stone columns. The Priebe method is the most widely used and was evaluated using the concept of reliability. Case histories with unsatisfactory performance allowed the development of application, design, and construction considerations for future projects and represent a summary of lessons learned from previous projects. A well-documented case history for an embankment widening project that used stone columns allowed an evaluation of current Federal Highway Administration recommendations for estimating settlements. Numerical analysis of this case history provided design guidance regarding the stress distribution in the soils underlying the stone column reinforced zone to use in the analysis of settlements.

CHAPTER 1. GENERAL INTRODUCTION

Geoconstruction technologies are utilized on transportation projects across the United States (U.S.) every day. Geoconstruction technologies include solutions for pavements, foundations, slopes, and retaining walls. From this broad field, two topic areas were developed to address knowledge gaps. First, an information system was developed to assist transportation engineers in geoconstruction technology decision making. Second, the performance of ground stabilized by stone columns was evaluated using case histories.

Web-Based Information System

Transportation engineers, planners, and officials lack a readily available system to access critical information with regard to geoconstruction technologies and assist in deciding which technologies are potentially applicable to their projects. To address this deficiency, a web-based information and guidance system was developed. The objectives of the system were (1) to provide an interactive information system that contains a technology catalog, technology selection assistance, and a glossary; and (2) to provide a selection system to develop a "short-list" of applicable technologies based on project and site characteristics.

The web-based information system contains the vital information for 46 geoconstruction technologies. The information contained in the system allows for selecting, applying, designing, cost estimating, specifying, and monitoring geoconstruction technologies. The information system is a comprehensive toolkit of geotechnical information to address all phases of decision making, from planning to design to construction, to allow transportation projects to be built faster, to be less expensive, and/or to last longer. Anyone involved in planning, design, and construction of transportation infrastructure will benefit from the information and resources contained in the system. The target audience of the information and guidance system is public agency personnel at the local, state, and federal levels. Other users may include engineering consultants, contractors, architect/engineer groups, and academics/students. The information system should enable the user to determine where, when, and how a certain geoconstruction technology should be used (Terrel *et al.* 1979).

Performance of Ground Reinforced by Stone Columns

Evaluating methods of estimating the performance of ground reinforced by stone columns included a thorough review of published literature focusing on case histories. The estimation of settlements for stone column reinforced ground was the initial study area because a standard design procedure for accurately estimating settlements has not been adopted in the U.S. The literature over the last two decades is clear on the inadequacies of current settlement prediction methods. For example, Allen *et al.* (1991), which included geotechnical engineers with the Washington State Department of Transportation, stated that "improvements in the semi-empirical settlement prediction methods involving stone columns are needed." Clemente and Davie (2000) found that "the results from full-scale tests show more improvement than predicted by theoretical procedures, although a large scatter was observed." Abdrabbo and Mahmoud (2002) stated that "there is no reliable procedure for settlement calculation of improved geomaterial by stone columns." Raman (2006) found measured settlements in stone column treated areas to be an average of 42 percent of the predicted settlements.

The Priebe method of estimating settlements of stone column reinforced ground was evaluated using data from case histories. The evaluation incorporated the concept of reliability. Considerations for future stone column projects were developed from case histories with both satisfactory and unsatisfactory performance. A feature of these considerations is the compilation of lessons learned from case histories with unsatisfactory performance. A well-documented case history was selected to evaluate the Equilibrium and Priebe methods of estimating settlements. Numerical modeling of the stress distribution below the stone column reinforced zone provided design guidance for future projects. This combination of focus areas will contribute to the proper application of the stone column geoconstruction technology and to prediction of the deformations associated with stone column reinforced ground.

Dissertation Organization

This dissertation is a compilation of four papers submitted, or to be submitted, to scholarly journals. A total of seven chapters comprise the dissertation. Chapter 1 provides a general introduction. Chapter 2 provides a detailed literature review for both the information system and the performance of stone columns that supplements the short literature reviews contained in the journal papers presented in Chapters 3 through 6.

Chapters 3 and 4 describe the web-based information system. The first paper included as Chapter 3 describes the structure and programming of the information system developed for 46 geoconstruction technologies. The technology-specific information available through the system for each technology is outlined. The information system contains technology selection assistance, which is detailed in the paper presented in Chapter 4. Chapter 4 describes the development and organization of the knowledge base that supports project-specific selection assistance for geoconstruction technologies.

Chapters 5 and 6 address the performance of stone column reinforced ground. An evaluation of the Priebe method for estimating settlements using a reliability framework is presented in Chapter 5. Lessons learned from previous projects and an evaluation of settlements for a specific case history are presented in Chapter 6 to provide considerations for future projects that utilize stone columns.

General conclusions and recommendations for further study are provided in Chapter 7.

References for Chapters 1, 2, and 7 and Appendix A are provided in the Bibliography that follows Appendix A. References for the journal papers in Chapters 3 through 6 are provided at the end of each chapter in the format specified by the journal.

CHAPTER 2. REVIEW OF LITERATURE

A review of existing literature for both the information system and the performance of stone columns is summarized in this chapter. Both topic areas required a broad literature review, which has been subdivided into different aspects of the topic areas.

Information System Literature

A literature review was completed to identify similar reports and systems previously developed for geoconstruction technologies. During the completion of this review, three different concepts emerged that are explained below. First, literature that focuses on previously programmed systems for geoconstruction technologies is presented. Second, literature discussing the process of selecting and applying geoconstruction technologies within the overall project context is summarized. Third, literature describing the geotechnical design process and the implementation of a geoconstruction technology is reviewed.

The literature search revealed the commitment of the research sponsor, the Transportation Research Board, to compiling and disseminating information regarding problem foundations for highway embankments. In 1966, Highway Research Record Number 133 contained five reports under the heading of “Utilization of Sites with Soft Foundations.” From this record, Moore (1966) summarized the New York State Department of Public Works procedures for dealing with foundation problems. In 1975, National Cooperative Highway Research Program (NCHRP) Synthesis of Highway Practice 29, *Treatment of Soft Foundations for Highway Embankments*, provided the first comprehensive review of the design process philosophy, treatment methods, special considerations, subsurface investigation and testing, and foundation treatment design (Johnson 1975). In

1989, NCHRP Synthesis of Practice 147, *Treatment of Problem Foundations for Highway Embankments*, expanded the 1975 Synthesis to include more treatment methods and included a section on construction and performance monitoring (Holtz 1989).

Previously Programmed Systems

Automated systems for various aspects of geotechnical engineering were found during the study. Toll (1996b) reviewed systems that have been developed for geotechnical applications. By 1996, over 103 knowledge-based applications had been developed in the field of geotechnical engineering (Toll 1996a). Knowledge-based systems make use of heuristics and separate the programming from the “knowledge” such that the programming does not change each time the “knowledge” is updated (Toll 1996a). Previous systems included expert systems, decision support systems, knowledge-based systems, and neural network approaches for the following areas of geotechnical engineering: site characterization, site investigation planning, interpreting ground conditions, soil classification and parameter assessment, rock classification and parameter assessment, conceptual design of foundations, detailed foundation design, pile driving, foundation construction, foundation problems, soil slopes, rock slopes, earth retaining structures, tunnels and underground openings, mining, liquefaction, ground improvement, geotextiles, groundwater/dams, and roads and earthworks. Rule-based systems dominated the earlier systems, with more complex systems being developed more recently. The previously programmed systems described in this section are presented in chronological order.

Improve

Chameau and Santamarina (1989) described the knowledge-based system, *Improve*, for the selection of soil improvement methods. The system approaches the process of selection as a classification problem similar to soil classification and mineral identification. The system uses a knowledge representation structure based on “windows” together with a best-first search algorithm. A “window” refers to a possibility number that characterizes an object with respect to the variable of interest and is a fuzzy set. The search algorithm includes a pre-processor, classification system, case-based system, and post-processor. The pre-processor collects the required input to form a stack of windows and then compares the input stack to the windows stack with each technology. An acceptability value is determined from this comparison to identify the most suitable technologies. Over 40 technologies were considered in the system, as presented in Table 1. The project-specific questions utilized to sort the geoconstruction technologies are included in Table 2. The knowledge in the system was acquired from Dr. Robert D. Holtz. Dr. Holtz also provided performance feedback, which resulted in a systematic consideration of technical limitations of the possible methods. However, common practice does pose some constraints on the applicability of a given method (Chameau and Santamarina 1989).

Chameau and Santamarina (1989) also noted that a geotechnical expert’s comprehension of a problem is affected by a large number of factors, including factors that are case-specific, context dependent, and subjective. Geotechnical experts make decisions based upon the recollection of previous cases, which is very relevant in geotechnical engineering where an emphasis is placed on experience. Systems such as *Improve* can help bring the state of the art to practice and to train professionals, recognize gaps in knowledge, and transfer the knowledge and accumulated experience of a few to a large number of practitioners (Chameau and Santamarina 1989). Soil improvement can be readily distilled into a decision support system because it is a well-defined domain, the selection of methods is well documented by the job characteristics and the required soil improvement, documented cases exist, and qualitative variables enter the decision process (Chameau and Santamarina 1989). The *Improve* system could not be located during this review.

Table 1. Geoconstruction technologies in *Improve* (Chameau and Santamarina 1989)

Geoconstruction technologies	
Densification Blasting	Electrokinetic injection
Blasting and vibratory rollers	Jet grouting
Vibratory probe	Remove and replace
Vibratory probe and vibratory rollers	Admixture stabilization
Vibrocompaction	Displacement blasting
Vibrocompaction and vibratory rollers	Prewetting loess
Compaction piles	Prewetting swelling clay
Heavy tamping	Structural fill
Heavy tamping and vibratory rollers	Lightweight fill
Vibratory rollers	Mix-in-place piles
Preloading	Mix-in-place walls
Preloading and drains	Heating
Surcharge fills	Freezing
Surcharge fills and drains	Stone columns
Dynamic consolidation	Root piles
Electro-osmosis	Soil nailing
Drains	Strip reinforcement
Particulate grouting	Moisture barriers
Chemical grouting	Geotextiles
Pressure injected lime	Berms
Displacement grout	

Table 2. ***Improve*** **project specific inputs (Chameau and Santamarina 1989)**

Project specific inputs	
Type of project	Relative density
Environmental freedom	Saturation conditions
Time available	Stratum (covered or uncovered)
Importance of increasing strength	Stage (built or not built)
Importance of reducing deformation	Is surface above water?
Importance of modifying permeability	Is surface treatment possible?
Position (depth) of layer	Is layered construction possible?
Distance to the neighbor/layer depth	Duration of improvement (permanent or temporary)
Structure width/layer depth	Equipment particular to each alternative
Special soil type	Materials required by each method
Particle size	

Expert System for Preliminary Ground Improvement Selection

Motamed, Salazar, and D'Andrea (1991) developed an Expert System for Preliminary Ground Improvement Selection (*ESPGIS*), which is based on a knowledge based expert system (KBES). The system is menu driven and can advise the user in selecting a ground improvement method or evaluate the user's preselected method. Motamed *et al.* (1991) indicate that KBES applications have been implemented in all areas of civil engineering, with 76 operational prototype expert systems reported by 1987. Ground improvement in the U.S. has not been fully accepted as common practice due to the nature of the construction industry, resulting in a slow transfer of technology from the specialty contractor to the designer. A time lag in the range of 5 to 10 years exists between the introduction of a method and the subsequent widespread acceptance (Motamed *et al.* 1991).

The development of the *ESPGIS* system is presented in five stages, as illustrated in Figure 1(Motamed *et al.* 1991). First, the problem is defined conceptually, the user group is defined, and the need for an expert opinion is documented. Second, the problem is accurately defined. Third, the knowledge base is acquired from experts and other knowledgeable sources. Fourth, a tool is selected based on the requirements of the problem domain. Fifth, coding and testing of the system is completed.

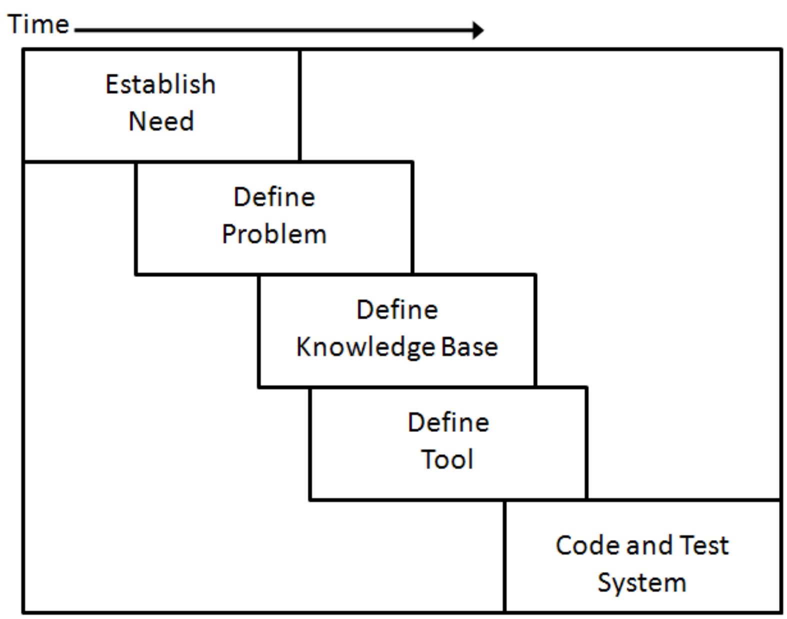

Figure 1. Stages in building a KBES (after Motamed *et al.* 1991)

The preliminary selection of ground improvement methods is not performed until the need for such modification is realized. The preliminary selection is based on the nature of the improvement and on physical subsurface, surface, and surrounding characteristics of the site. In developing the knowledge base for *ESPGIS*, published information and contractor's literature was used extensively. The methods included in *ESPGIS* are presented in Table 3. Geotechnical experts were not actively engaged in the development process. The selection of an expert system shell was shown to be important in the success potential of a KBES system (Motamed *et al.* 1991). The system was coded using VP-Expert in an MS-DOS based system. The components of the *ESPGIS* system are shown in Figure 2. The *ESPGIS* system could not be located during this review.

Table 3. Geoconstruction technologies in *ESPGIS* (Motamed *et al.* 1991)

Geoconstruction technologies	
Dynamic compaction	Slurry walls
Vibro-compaction	Diaphragm walls
Vibro-replacement	Chemical grouting
Compaction grouting	Slurry grouting
Pre-loading	Freezing
Wick drains	Jet grouting
Ground anchors	Lime injection
Mini-piles	

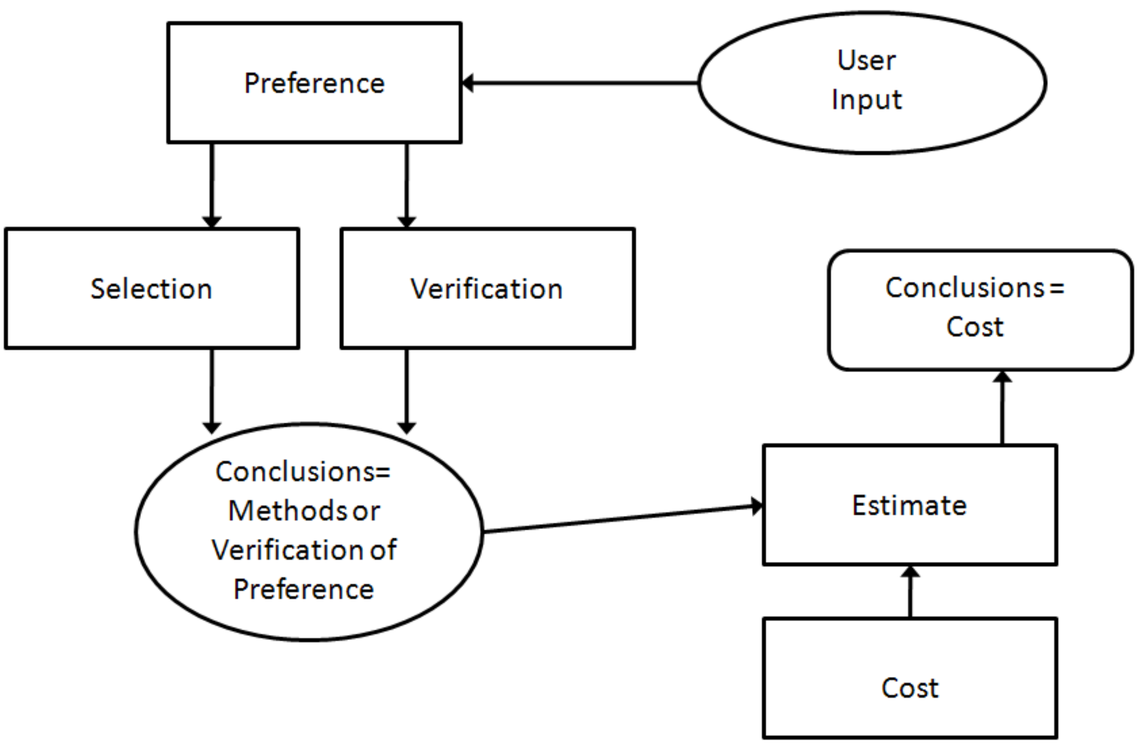

Figure 2. Components of ESPGIS (after Motamed *et al.* 1991)

International Knowledge Data Base for Ground Improvement Geo-Systems

Yoon, Thevanagayam, and Juran (1994) developed an International Knowledge Data Base for Ground Improvement Geo-Systems (*IKD-GIGS*), which was to aid rational selections, design, and construction of ground improvement technologies. DiMillio (1999) in *A Quarter Century of Geotechnical Research* stated that Federal Highway Administration (FHWA) joined forces with the International Center for Ground Improvement Technology in Brooklyn, New York, to develop this system. This system was intended to provide a comprehensive, user-friendly database from which a user could retrieve information on possible technologies by viewing similar case histories, problems encountered, possible remedial action schemes, comparative cost data, specifications and codes, and quality control/quality assurance (QC/QA). The ground improvement technologies included in *IKD-GIGS* are shown in Table 4. The system was programmed using a DOS-based system to facilitate the program operating on a personal computer. A relational database system was selected to implement *IKD-GIGS* because the software was economical, popular, powerful, and easy to use. The database included a compendium of national and international codes of practice, a collection of monitored case histories, and information on instrumented structures. As of 1999, the system contained more than 200 documented records of ground improvement case histories from 15 countries. Yoon *et al.* (1994) described the initial phase of work and indicated that the *IKD-GIGS* system was to be developed in multiple phases. The *IKD-GIGS* could not be located during this review.

Table 4. Geoconstruction technologies in *IKD-GIGS* (Yoon *et al.* 1994)

Ground improvement technologies	Ground reinforcement technologies	Ground treatment technologies
Dynamic Consolidation	Reinforced soils	Compaction
Vibrocompaction	Geosynthetics	Jet
Vacuum consolidation	Fiber reinforcement	Permeation
Drainage	Texsol	Hydrofracture
Preloading	Mechanically stabilized embankments	Compensation
Blasting	Anchorages	Fissure
Heating	Nails	Bulk
Freezing	Pinpiles	Slabjacking
Stone and lime columns	Diaphragm walls	Deep soil mix
Electro-chemical treatment		Shallow soil mix

Soil and Site Improvement Guide

Sadek and Khoury (2000) developed a selection system as part of a specialized geotechnical engineering soil improvement course at the American University of Beirut. The main objective of the system was to enhance the quality of the teaching and learning process as it relates to soil improvement. The end product provided a system for learning about different techniques, their advantages and limitations, their applicability under certain conditions, and the associated costs. Seventeen ground modification methods were included in the program and broken into four categories, as shown in Table 5. The *Soil and Site Improvement Guide* software presents the user with a series of modules that utilize an interface developed with Microsoft Visual Basic and a knowledge base in a Microsoft Access database (Sadek and Khoury 2000). The *Soil and Site Improvement Guide* could not be located during this review.

Table 5. Geoconstruction technologies in *Soil and Site Improvement Guide* (Sadek and Khoury 2000)

Densification methods	Adhesion methods	Reinforcement methods	Physicochemical methods
Dynamic deep compaction	Cement grouting	Minipiles	Electro-osmosis
Surcharging	Chemical grouting	Soil nailing	Lime treatment
Vibrocompaction	Slurry grouting	Soil and rock anchors	Soil mixing
Vibroreplacement	Freezing		Vitrification
Compaction grouting			
Accelerated consolidation/wick drains			

Summary of Existing Systems

Although the previously developed systems would have been very beneficial to the development of the information system described in Chapters 3 and 4, none of the identified systems could be accessed during this review. The framework and logic from the previously developed systems were considered in the development of the web-based information system described in Chapters 3 and 4. A knowledge gap exists due to the failure of previously developed systems to be maintained, updated, and publicly available.

Geotechnical Design Process Review

The information system is applicable to a very wide range of projects ranging from embankments to retaining walls to pavement foundations. Each project has a unique design process. The literature identified in this section provides some background to the geotechnical design process required for the various project types. The queries in the selection system and how the selection system will be used by a practicing engineer required consideration during development of the overall information system.

Treatment of Problem Foundations for Highway Embankments

Holtz (1989) developed a list of questions, summarized in Table 6, that begins the process of evaluating project conditions and geoconstruction technologies. Table 7 describes some of the factors involved in constructing embankments on problem soils. Figure 3 illustrates the typical process of incorporating geotechnical information into project planning.

Table 6. Questions involved in highway construction on problem foundations (Holtz 1989)

Question	Additional queries
Elevated structure or embankment?	Will the embankment be stable? What is the probability and cost of failure? Can an embankment provide a satisfactory riding surface? Can added cost of elevated structure be justified? How much time is available for construction? What are relative maintenance costs? What is the economic/design life of the structure?
Can, or should, postconstruction embankment settlements be accepted?	Will settlements be uniform or irregular? Should design remove all primary settlements and reduce secondary compression settlements?

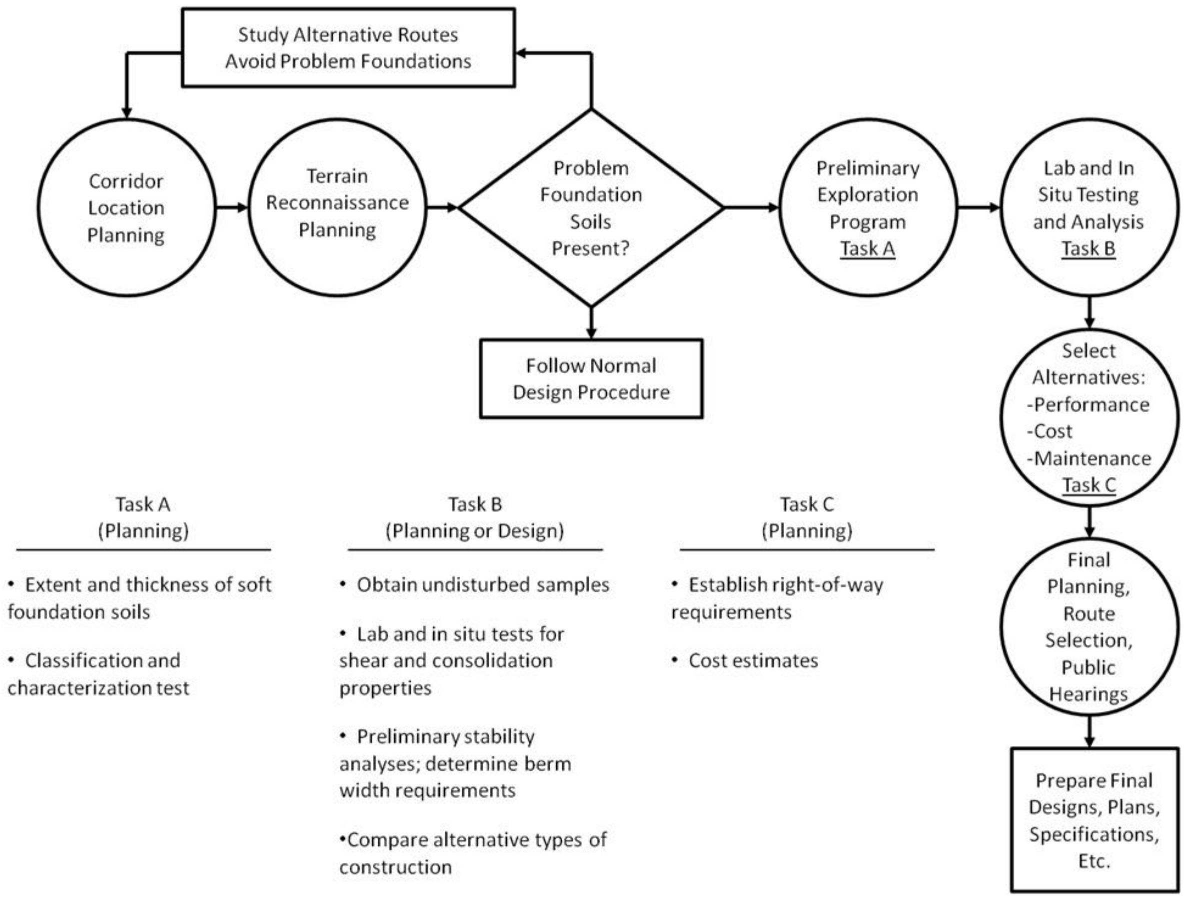

Figure 3. Requirements for input of geotechnical information into the corridor planning phase when problem soils are present (after Holtz 1989)

Preliminary Ground Improvement Selection

Beyond the intricacies of the expert system, the overall ground improvement process is discussed below and divided into four parts, as shown in Figure 4 (Motamed *et al.* 1991). The four parts are geotechnical study and evaluation, design and performance predictions, performance of ground improvement, and project evaluation. The geotechnical study and evaluation is typically conducted by the geotechnical engineer and the specialty contractor. Design and performance predictions are prepared if ground improvement is required. At this stage, the specialty contractor prepares detailed designs, work plans, schedules, and estimates. Once construction begins, the process is measured by previously set or established quality control criteria. Project evaluation is the degree of conformance of the ground's performance to the required performance and often includes testing of the ground (Motamed *et al.* 1991).

Table 7. Factors involved in constructing embankments on problem foundations (Holtz 1989)

Item	Remarks
Additional construction costs	Substantial; may be as much as several million dollars per mile.
Safety and public relations	Excessive post-construction differential settlements may require taking part of roadway out of service for maintenance: Serious safety hazard for heavily traveled roads. Major inconvenience—public relations problems.
Maintenance cost	May be large: • More expensive construction may minimize post-construction maintenance. • Maintenance costs are sometimes regarded as deferred construction costs.
Environmental considerations	May determine type of highway construction and possible alternatives for foundation treatment.
Foundation stability during construction	Detailed subsurface investigations, laboratory and in situ tests, and design studies required.
Tolerable postconstruction total and differential settlements	Appropriate criteria not well formulated; subjective; depends on engineering and public attitudes.
Structure vs. embankment	An important decision affecting both construction and maintenance costs.
Construction time available	Some alternatives may be eliminated by need for early completion date.

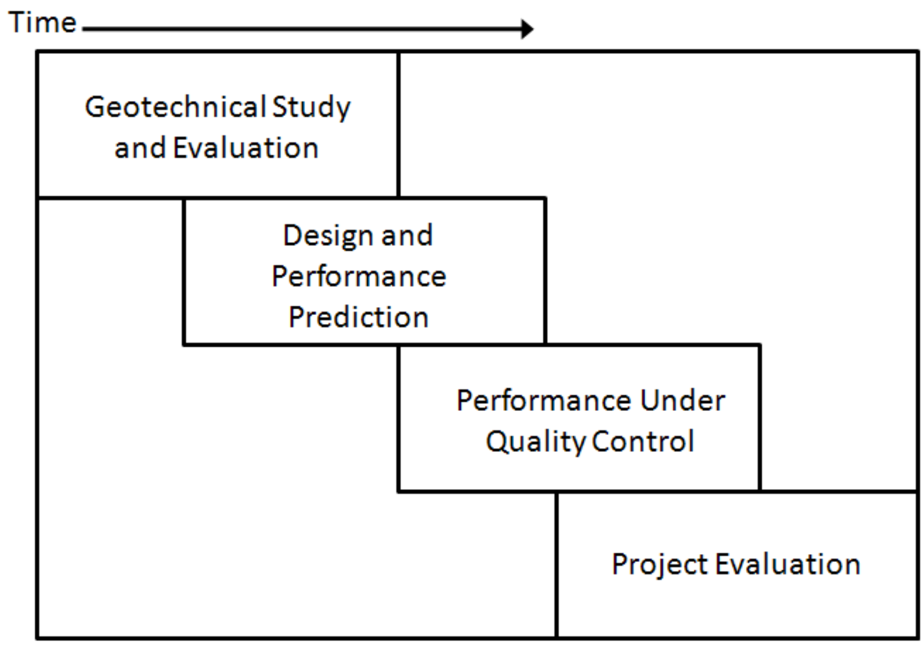

Figure 4. Stages of a ground improvement project (after Motamed *et al.* 1991)

Guidelines on Ground Improvement for Structures and Facilities

The U.S. Army Corps of Engineers described factors to consider in assessing, designing, and selecting which technique(s) to utilize for a particular project (Dept. of the Army 1999). The first area discussed is described as Design Considerations and Parameters and considers site constraints, subsurface conditions, scheduling, budget, and availability of contractor. The second area is described as Design Procedures and included the following steps:

1. Select potential improvement methods.
2. Develop and evaluate remedial design concepts.
3. Choose methods for further evaluation.
4. Perform final design for one or more of the preliminary methods.
5. Compare final designs and select the best one.

6. Field test for verification of effectiveness and development of construction procedures.
7. Develop specifications and QC/QA programs.

Soil Improvement

Holtz *et al.* (2001) in a handbook chapter discussed the following nine factors to consider in assessing which technique(s) may be the most appropriate:

1. Operational criteria for the facility.
2. Area, depth, and total volume of soil to be treated.
3. Soil type and its initial properties, depth to water table.
4. Availability of materials.
5. Availability of equipment and required skills.
6. Construction and environmental factors, such as site accessibility and constraints.
7. Local experience and preference, politics and tradition.
8. Time available.
9. Cost.

Key Elements in Deep Vibratory Ground Improvement

Bell (2004) discussed the importance of the construction technique in regard to deep vibratory ground improvement. Bell (2004) stated, “Deep vibratory ground improvement is best understood as a process rather than a product. It can be applied most effectively if all the elements of the process are understood in relation to each other, and if each is given proper attention at all stages.” The sequence set forth is apparently chronological but this may not always be the case. The following key elements are identified in the selection and implementation process:

1. Site evaluation
2. Ground investigation
3. Development of concept
4. Design
5. Construction technique
6. Process evaluation
7. Commissioning and maintenance

Some Applications of Ground Improvement Techniques in the Urban Environment

Serridge (2006) developed Figure 5 to describe the key aspects for achieving a successful ground improvement project. Steps for achieving a successful project are provided for each phase of implementation.

Ground Improvement Methods

Elias *et al.* (2006) described the following sequential process for the selection of candidate ground improvement methods for any specific project. The steps in the process include evaluations that proceed from simple to more detailed, allowing a best method to emerge. The process is described as follows:

1. Identify potential poor ground conditions, their extent, and type of negative impact.

2. Identify or establish performance requirements.
3. Identify and assess any space or environmental constraints.
4. Assessment of subsurface conditions.
5. Preliminary selection.
6. Preliminary design.
7. Comparison and selection.

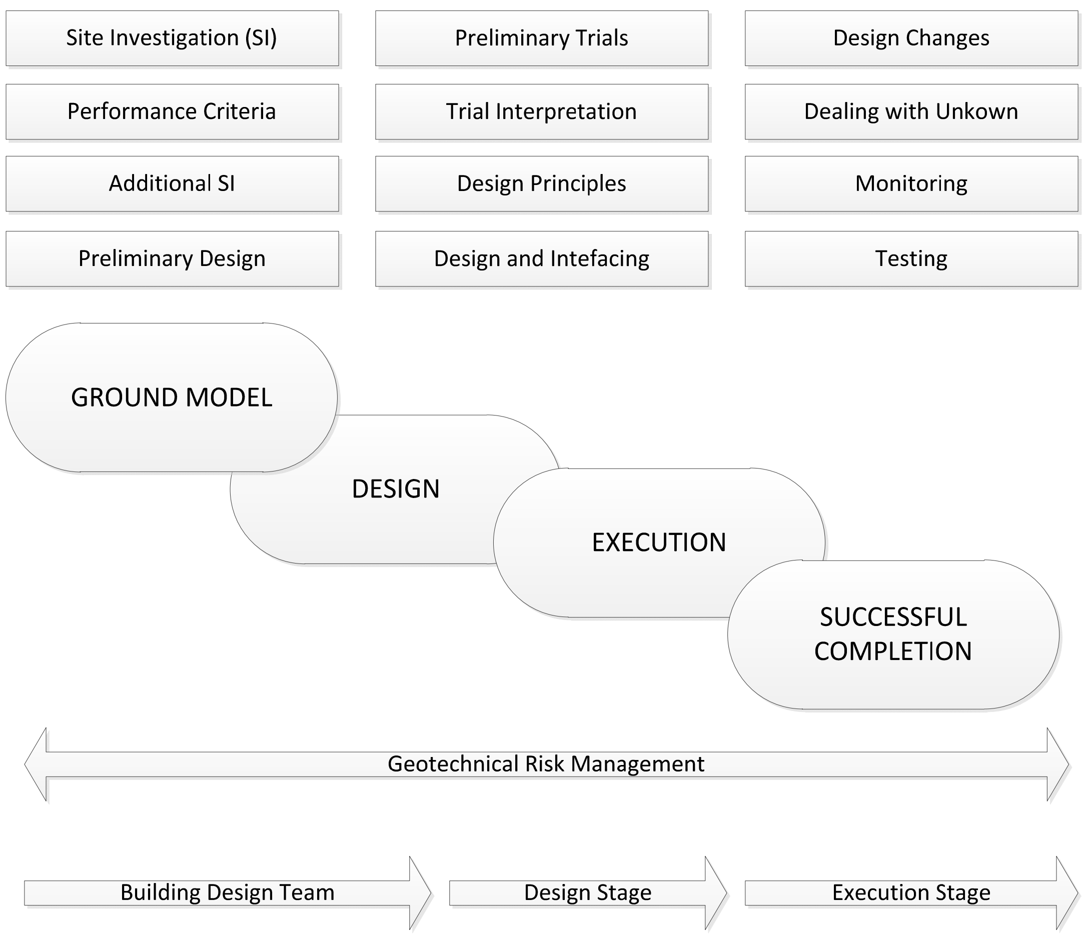

Figure 5. Steps for achieving successful ground improvement implementation (after Serridge 2006)

Geosynthetic Design and Construction Guidelines

Holtz *et al.* (2008) presented the following steps for design of a reinforced soil slope:

1. Establish the geometric, loading, and performance requirements for design.
2. Determine the subsurface stratigraphy and the engineering properties of the in-situ soils.
3. Determine the engineering properties of the available fill soils.
4. Evaluate design parameters for the reinforcement (design reinforcement strength, durability criteria, soil-reinforcement interaction).
5. Determine the factor of safety of the unreinforced slope.
6. Design reinforcement to provide stable slope.
7. Select slope face treatment.
8. Check external stability.
9. Check seismic stability.
10. Evaluate requirements for subsurface and surface water control.
11. Develop specifications and contract documents.

Geotechnical Aspects of Pavements

Christopher *et al.* (2010) outlined two procedures for utilizing geosynthetic reinforcement for base reinforcement and stabilization. The following design approach is for base reinforcement using geosynthetics, which is summarized from American Association of State Highway and Transportation Officials (AASHTO) 4E.

1. Initial assessment of applicability of the technology.
2. Design of the unreinforced pavement.

3. Definition of the qualitative benefits of reinforcement for the project.
4. Definition of the quantitative benefits of reinforcement through the Traffic Benefit Ratio or Base Course Reduction Ratio.
5. Design of the reinforced pavement using the benefits defined in Step 4.
6. Analysis of life-cycle costs.
7. Development of a project specification.
8. Development of construction drawings and bid documents.
9. Construction of the roadway.

Christopher *et al.* (2010) also outlined the design of the geosynthetic for stabilization using the design-by-function approach in conjunction with AASHTO M288, in the steps from FHWA HI-95-038 (Holtz *et al.* 1998) outlined below. A key feature of this method is the assumption that the structural pavement design is not modified at all in the procedure. A limited summary of the procedure outlined in Christopher *et al.* (2010) is outlined below.

1. Identify properties of the subgrade, including California Bearing Ratio (CBR), location of groundwater table, AASHTO and/or Unified Soil Classification System (USCS) classification, and sensitivity.
2. Determine if a geosynthetic will be required.
3. Design the pavement without consideration of a geosynthetic, using normal pavement structural design procedures.
4. Determine the need for additional imported aggregate to improve mixing at the base/subgrade interface. If such aggregate is required, determine its thickness, t_1, and reduce the thickness by 50%, considering the use of a geosynthetic.

5. Determine additional aggregate thickness, t_2 , needed for establishment of a construction platform. The FHWA procedure requires the use of curves for aggregate thickness vs. the expected single tire pressure and the subgrade bearing capacity.
6. Select the greater of t_2 or 50% t_1.
7. Check filtration criteria for the geotextile to be used. For geogrids, check the aggregate for filtration compatibility with the subgrade, or use a geotextile in combination with the grid to meet the project requirements.
8. Determine geotextile or geogrid survival criteria. The design is based on the assumption that the geosynthetic cannot function unless it survives the construction process.

Ground Improvement – Principles and Application in Asia

Raju (2010) provided a few factors to consider in the important decision of choosing which ground improvement method to utilize, as listed below. The reference provides additional discussion for each of the following factors:

1. Suitability of the method.
2. Technical compliance.
3. Availability of QC/QA methods.
4. Availability of material.
5. Time.
6. Cost.
7. Convenience.

8. Protection of the environment.

Summary of Geotechnical Design Process Review

The web-based information system covers a broad range of geoconstruction technologies. Each technology has a unique application and design process. A consideration in the development of the web-based information system was determining how a user integrates the information system into the design process. To address this consideration, a webpage titled Geotechnical Design Process was developed. Technology-specific design considerations were included in the documents available for download for each technology, as described in Chapter 3.

Geoconstruction Technology Application and Selection

Literature regarding the application and selection of geoconstruction technologies was found in published papers and reports. Table 8 summarizes the abundant literature that addresses identifying and applying geoconstruction technologies. The identified geoconstruction technology information is presented in chronological order. The literature summarized in Table 8 was utilized in the development of the technology selection assistance provided as part of the web-based information system.

Table 8. Geoconstruction technology application and selection literature summary

Reference	Topic	Comment
Terrel *et al.* 1979	Soil stabilization in pavement structures	Guidelines for the selection and application of soil stabilizers for pavements.
Holtz 1989	Treatment of problem foundations for highway embankments	Presents a summary of foundation treatment methods grouped in the following five areas: reducing the load, replacing the problem materials by more competent materials, increasing the shearing strength and reducing compressibility of the problem materials, transferring the loads to more competent layers, and reinforcing the embankment and/or its foundation
Bergado *et al.* 1996	Soft ground improvement in lowland and other environments	Detailed information for surface compaction, deep compaction, prefabricated vertical drains, granular piles, lime/cement stabilization, and mechanically stabilized earth. Flow charts to guide in the selection of the ground improvement technique for shallow ground improvement and deep ground improvement also presented.
Van Impe *et al.* 1997a	Soil improvement in belgium	An overview of the state of the art in Belgium for 20 geoconstruction techniques. A unique aspect of the overview is that the various types of technologies were summarized as to the frequency of use, which were regular, sporadic, seldom and never.
Dept. of the Army 1999	Guidelines on ground improvement for structures and facilities	The guidance addresses planning, site evaluation, determining if ground improvement is required, selection of improvement method, cost, design, construction, and performance evaluation. A series of 26 figures, most with flow charts, are presented for determination of the need for ground improvement. A listing of potentially applicable ground improvement methods for civil works structures are shown for various constraints and goals.

Table 8. (continued)

Reference	Topic	Comment
Berg *et al.* 2000	Geosynthetic reinforcement of the aggregate base/subbase courses of pavement structures	The application of various types of geosynthetic reinforcement for permanently paved roads is shown for a range of subgrade strengths and base/subbase thicknesses.
Holtz *et al.* 2001	Soil improvement	This handbook chapter provides the properties of many soil improvement methods according to classification and soil type.
Charles and Watts 2002	Treated ground: engineering properties and performance	Detailed introduction to the process of evaluating sites, implementing the treatment, and evaluating the effectiveness of the treatment. Three categories of ground treatment evaluated were improvement by compaction, improvement by consolidation, and improvement by stiffening columns. The technical adequacy of specific ground treatments is described.
Burke and Sehn 2003	Influence of ground improvement on geotechnical design	Both authors were employed by a geotechnical specialty contractor and acknowledge that "a vast amount of experience has developed, both in application as well as in performance, but much of the performance data remains unpublished or undocumented." The applicability of the ground improvement methods based on the project objectives, technical decision considerations, and construction/cost issues are presented.
Elias *et al.* 2006	Ground improvement methods	This reference represents the latest FHWA ground improvement manual. The manual contains various categories, functions, methods, and applications for ground improvement technologies. Comparative costs for the ground improvement methods are presented.

Table 8. (continued)

Reference	Topic	Comment
Holtz *et al.* 2008	Geosynthetic design and construction guidelines	A 30+ year history of successful use of geosynthetics for the stabilization of very soft wet subgrades is described. A summary of the application and associated functions of geosynthetics in roadway systems is developed which culminates in the subgrade conditions which are considered optimum for using geosynthetics in roadway construction
Chu *et al.* 2009	Technical Committee 17 (TC 17) classification of ground improvement methods	The International Society for Soil Mechanics and Geotechnical Engineering TC 17 developed a classification of ground improvement methods. A systematic evaluation of various ground improvement methods to treat soft cohesive soils without admixtures is described. A summary of ground improvement methods for mitigation of liquefaction is based on Mitchell (2008).
Christopher *et al.* 2010	Geotechnical aspects of pavements	Guidelines are presented for various stabilization methods for pavement foundations, the general function and typical application of geosynthetic usage in transportation, the appropriate subgrade conditions for stabilization using geosynthetics, the selection of admixture stabilization method(s), and recommended field compaction equipment for different soil types based on Rollings and Rollings (1996).
Raju 2010	Ground improvement – principles and application in asia	An overview of the practice in Asia. The practice in Asia is typically in one of the following four areas of ground improvement: (1) consolidation (e.g. prefabricated vertical drains & surcharge, vacuum consolidation, and stone columns); (2) chemical modification (e.g. deep soil mixing, jet grouting, injection grouting); (3) densification (e.g. vibrocompaction, dynamic compaction, compaction grouting); and (4) reinforcement (e.g. stone columns, geosynthetic reinforcement).

Performance of Stone Columns Literature Review

The literature review identified case histories relating to stone columns and methods of estimating settlements. Fundamentals, assumptions, and factors influencing the estimation of the performance of stone column reinforced ground are also included. The following review is intended to supplement the material presented in Chapters 5 and 6.

Case Histories

A large number of case histories were identified, with each case history varying significantly with regard to site conditions, design details, construction methods, QC/QA, and settlement monitoring during construction. The literature review targeted case histories that reported data relevant to the settlement performance of stone column reinforced ground. Although the study was focused on settlements of stone column reinforced ground, the case histories yielded information regarding other aspects of stone column construction, such as installation effects, vibrations, and sustainability.

In order to condense the case histories, case histories with satisfactory performance have been sorted according to site conditions and presented in chronological order according to the date of the published reference. Each table provides the reference(s), project and location, soil conditions, and some brief comments regarding the specifics of the case history. The case histories have been sorted by site conditions into different tables, which are included as part of the journal paper in Chapter 5 and presented in Appendix A. Case histories applicable to more than one site condition have been included in multiple tables as appropriate.

Case Histories with Long-Term Settlement Monitoring

Case histories with long-term monitoring to capture the secondary compression behavior of stone column reinforced sites provide critical information with regard to estimating total long-term settlements. Case histories typically only captured primary consolidation or what the researchers/authors considered to be elastic settlements. These case histories are highlighted, as the data obtained is vital for a complete understanding of the settlement characteristics of stone column reinforced ground. For reference to the secondary compression coefficients reported in Table 9 for projects with peat, the values tend to agree with Christopher and Wagner's (1988) findings of 0.03 for fibrous peat and 0.01 for sedimentary peat at an untreated project near West Bend, Wisconsin. The review indicates a lack of case histories that monitor settlements over a sufficient time frame to capture the portion of the settlement attributable to secondary compression.

Table 9. Summary of case histories with long-term settlement monitoring

Reference	Time settlements monitored (days)	Settlement occurring at end of monitoring period? (yes/no)	Coefficient of secondary compression, C'_α	Comments
Colleselli *et al.* 1983	~900 (after building construction)	Yes	Not reported	Improved sands, silts, and clays. Although mostly granular materials, some settlement continued to occur at 2 to 3 years after loading.
Waterton and Foulsham 1984	~200 (after reaching finished grade)	No	0.04	Based on back analysis of embankment monitoring at a mangrove mud site in Darwin, Australia.

Table 9. (continued)

Reference	Time settlements monitored (days)	Settlement occurring at end of monitoring period? (yes/no)	Coefficient of secondary compression, C'_α	Comments
Greenwood 1991	~100 (after full loading)	Yes	Not reported	Canvey Island.
	~200 (after reaching finished grade)	Yes	Not reported	Humber Bridge – load, settlement, and pore pressure plots provided.
Cooper and Rose 1999	~200 (after reaching finished grade)	Very close to end of secondary compression	0.02 to 0.03 for area with peat layer, but found as high as 0.05	Based on back analysis of embankment monitoring.
Raju 1997	~100 (after reaching finished grade)	No	Not reported	Kinrara project – 90% consolidation at 90 days after reaching finished grade.
	~225 (after reaching finished grade)	No	Not reported	Kebun project – 90% consolidation at 180 days after reaching finished grade.
Raju *et al.* 2004	~250 (after reaching finished grade)	No	Not reported	For 12 m (39 ft) high embankment, tin slime settlement appeared elastic in nature with no long-term settlement.
Bhushan *et al.* 2004	~180 (after reaching finished grade)	Very close to end of secondary compression	Not reported	90% consolidation within 30 to 45 days after reaching finished grade.
Oo 2004	~100 (after reaching finished grade)	No	Not reported.	Excess pore pressures dissipated at 90-100 days.
Clemente and Parks 2005	~300 (after loading)	No	Not reported	Mostly granular soils, yet settlement continued to occur for close to a year in the granular soils.

Case Histories with Rate of Consolidation

Stone columns increase the rate of consolidation by providing a drainage path for water within the soil. Although not a focus in this study, case histories with reported values for the rate of consolidation are included below as the time for consolidation is a design consideration when stone columns are utilized to reinforce weak fine-grained soils.

- Waterton and Foulsham (1984) back calculated a coefficient of vertical consolidation, c_v, of 1.1 m^2/yr (12 ft^2/yr) for a stone column project.
- De Silva (2005) described a land reclamation project in Hong Kong. The c_v (for virgin compression) determined from standard consolidation testing ranged from 0.6 to 2 m^2/yr (6 to 22 ft^2/yr) with an average value of about 1 m^2/yr (11 ft^2/yr). The field coefficient of horizontal consolidation, c_h, as determined from settlement markers, ranged from 2.8 to 4.0 m^2/yr (30 to 43 ft^2/yr) with an average of 3.5 m^2/yr (38 ft^2/yr).
- Raman (2006) detailed several sections of a project where stone columns were utilized to reinforce the foundation soils for a north-south expressway in Malaysia. Raman (2006) completed a back analysis compared to design rates of consolidation according to Han and Ye (2001). The field values of c_v ranged from 4.1 to 12 m^2/yr (44 to 130 ft^2/yr) and c_h ranged from 7.3 to 12.3 m^2/yr (79 to 132 ft^2/yr). On average, the c_h was about 1.6 times the c_v (Raman 2006).

Case Histories with Unsatisfactory Performance

Unsatisfactory performance, or problematic conditions, identified in the case histories that resulted in unsatisfactory stone column performance are summarized in Table 10. It should be noted that unsatisfactory performance in other conditions may have occurred and was not reported, or the ground improved might have performed satisfactorily without the use of stone columns (Charles and Watts 2002). The problematic conditions can be related to site conditions, design, construction, and/or QC/QA. Project considerations are provided for each problem condition.

Table 10. Documented problematic conditions

Problem condition	Reference (s)	Project considerations
Sensitive soils	McKenna *et al.* 1975; Wijeyakulasuriya *et al.* 1999; Gue and Tan 2003; Oh *et al.* 2007a; Oh *et al.* 2007b	Stone columns should be used with caution on projects with sensitive clays.
Thick peat deposits	Slocombe 2001	Peat layers have to be accommodated and considered in design and construction.
Quick, small-scale load test	Greenwood 1991; Chummar 2000	The scale of the load test should be representative of project conditions.
Loading rate did not allow dissipation of excess pore water pressures	Chummar 2000; Greenwood 1991	Analysis must consider reduction in strength of in situ soils upon loading. Construction should be overseen by experienced geotechnical engineer using data from piezometers and settlement plates.
Stone columns became fouled at surface and did not allow drainage	Chummar 2000	The stone columns should be directly connected to the drainage blanket and construction should not allow the tops of the stone columns to become fouled.
Ground disturbance adjacent to stone columns	Venmans 1998	Projects should include repair or replacement plans for items such as road signs which can be damaged by heaving ground.
Fill heterogeneity	Clemente and Davie 2000; Slocombe 2001	Even with stone columns, the variability of the fill can result in a very wide range of support conditions.
Collapsible soils or fills	Slocombe 2001; Charles and Watts 2002	Stone columns have the potential to supply water to the soils which can result in collapse.

Table 10. (continued)

Problem condition	Reference (s)	Project considerations
Lack of adequate geotechnical investigation	Meade and Allen 1985; Slocombe 2001; Charles and Watts 2002	A detailed geotechnical investigation is required for stone column projects. No areas of serious doubt should exist within the area to treat.
Stiffer soils encountered during construction which slowed installation	Meade and Allen 1985	
Lack of global stability considerations	Charles and Watts 2002; Gue and Tan 2003	Designers must consider all possible scenarios which affect a project site.
Lack of construction supervision by engineer of record	Gue and Tan 2003	QC/QA is essential to satisfactory performance of stone columns.
Poor construction methods	Bell 2004	
Installation in stiff soils	Chen and Bailey 2004	Stone columns installed into stiff to hard soils resulted in a weaker soil structure.
Lack of expected improvement at edge of treated area	Cooper and Rose 1999	A reduced efficiency of stone columns along the edges of a widely-reinforced area are should be anticipated.

Stone Column Installation Effects

Vibrated stone column installation methods have significant variation in performance as a result of the construction technique (Bell 2004). The methods of constructing stone columns developed from earlier construction practices that were utilized to densify clean, granular soils with depth. This section addresses the effect of installation on the in situ soil properties and the resulting stone column material strength. The case histories summarized in this section illustrate the lack of a consistent response of the in situ soils due to stone column installation. Specific equipment operating on a specific site using a specific installation method results in a unique effect on the in situ soil properties post-installation. No clear, accepted means of anticipating installation effects has been identified, but what is clear is that the installation effects influence the performance of the stone column treated ground (Egan *et al.* 2009).

The most commonly accepted installation effects are smearing of cohesive soils along the sides of the cavity during construction, which reduces the horizontal permeability of the system, and densification of granular soils associated with the construction vibrations, which in turn results in increased strength (Egan *et al.* 2009). The installation effects of stone columns are extremely complex and involve a series of loadings and unloadings, as well as vibration considerations. Installation effects are typically studied through evaluation of the horizontal to vertical stress ratio, K. K_o, the coefficient of lateral earth pressure at-rest, is defined as the ratio of the effective horizontal stress divided by the effective vertical stress, which is the in situ value of K prior to stone column installation. For normally consolidated soils, the Jaky expression K_o=1-sin Ø', where Ø' is the effective angle of internal friction for the soil, is typically used to estimate K_o (Das 1998). K_p is defined as Rankine's passive earth pressure coefficient and can be determined using K_p=tan^2 (45+Ø'/2). K* is termed the post-installation ratio of horizontal to vertical soil stresses. Elshazly *et al.* (2008b) summarized published K* values, as shown in Table 11. It should be noted that the references in Table 11 did not explicitly state whether K* was based on total or effective stresses. A summary of findings regarding lateral stress, lateral displacement, and ground surface heave from installation studies is provided in Table 12.

Table 11. Published K* values (after Elshazly *et al.* 2008b)

Reference	K* value	Method of determination
Goughnour 1983	Between K_o and $1/K_o$	Analytical solution based on elastic and rigid-plastic behavior using the unit cell concept.
Priebe 1995	1.0	Analytical solution of end-bearing incompressible columns.
Watts *et al.* 2000	Between K_o and K_p	Full-scale load tests on vibro-displacement stone columns in variable fill.
White *et al.* 2002; Pitt *et al.* 2003	Between 0.4 and 2.2, with average of 1.2	Full-scale load tests on vibro-displacement stone columns in compressible clays and silts underlain by highly weathered shale.
Elkasabgy 2005	Between 0.7 and 2.0, with average of 1.2	Back calculations from 3 full-scale load tests performed on stone columns within 3 extended arrays of columns.
Elshazly *et al.* 2006	Between 1.1 and 2.5, with best estimate of 1.5	Back calculations from full-scale load test performed on a stone column within an extended array of columns.

Table 12. Summary of installation studies

Reference	Study type	Findings
Watts *et al.* 2000	Full-scale instrumented field study	The pressure cell data showed that lateral stress increases in the surrounding soil during probe insertion and stone column construction.
White *et al.* 2002	Full-scale instrumented field study	A K_o Stepped-Blade device did not consistently show lateral stresses higher than the initial in situ tests pre construction. White *et al.* (2002) described ground heave and radial cracking during stone column construction, and hypothesized that the in situ soil strength could have been reduced due to disturbance.
Guetif *et al.* 2007	Numerical simulation	The numerical results indicated the surrounding soil to a distance equal to the diameter of the column will experience an increase in effective stresses due to construction.
Elshazly *et al.* 2008a	Numerical simulation	Model validated with a well-documented case history. Showed an increase in K* at column spacings less than 1 m (3.3 ft). Found that K* varied based on many factors, such as the type of installation equipment, its power and effective amplitude, as well as the soil type and the installation procedure.

Table 12. (continued)

Reference	Study type	Findings
Egan *et al.* 2009	Dry bottom-feed projects	Heave during installation is presented using the Heave Ratio which is defined as the volume of heaved ground divided by the total volume of stone columns. Project observations resulted in Heave Ratios ranging from zero for a two column group, to 27% for a 25-column group, to 75% for an infinite pattern of columns. The process of constructing stone columns results in lateral and vertical soil displacements, which are a function of the lateral confinement of the soil and adjacent stone columns.
Kirsch 2006, Kirsch 2009, Kirsch and Kirsch 2010	Full-scale instrumented field study	The study utilized column groups of 25 in silty clay and sandy silt soils. The results were reported as the ratio of K* divided by K_o. The maximum increase in K* occurred at a distance between 3 and 5 times the diameter from the stone column axis in both the silty clay and sandy silt soils. The maximum K* in the silty clays was generally 1.2 K_o to 1.3 K_o and for the sandy silts was in the range from 1.4 K_o to 1.6 K_o. A Menard pressure meter showed an increase in stiffness in the range of 100 to 250% at a distance between 3 and 6 times the diameter from the stone column axis for both soil types. Near the stone column, the post-installation sandy silt modulus was shown to be lower than the initial modulus, where for the silty clay the modulus was increased by more than 200% adjacent to the stone column.
Castro and Karstunen 2010	Numerical simulation	Simulated the results presented by Kirsch (2006) and found a similar trend with respect to ground stiffness in the soil surrounding the stone column.

The installation effects are not limited to the in situ soil. The influence of installation technique on the property of the stone column materials was studied by Herle *et al.* (2009). The friction angle of the stone column material, which is a common input in most settlement analyses, was shown to be dependent on the pressure level, or compaction effort, during installation. The study presented field measurements using dry density and void ratio of the stone column as proxies for strength of the stone column. The dry density of the stone column material was found to increase with depth and the void ratio was found to decrease with depth, which indicates that the stone column friction angle increases with depth. Herle *et al.* (2009) also found that during densification, grain crushing and segregation take place, which results in a decrease in the void ratio. The study found that most cases result in friction angles above 50° and that design methods should utilize a value in this range, as compared to conventional designs, which use a friction angle of approximately 40° (Herle *et al.* 2009).

Utilization of Case Histories

The case histories provided an overview of satisfactory performance and described many of the problems observed on past stone column projects. The case histories varied greatly in detail with regard to site conditions, soil parameters, loading conditions, construction details, QC/QA, and settlement monitoring. Installation effects were shown to have an impact on the performance of stone columns. However, consistent improvement of soil in terms of post-installation lateral stresses was not apparent. For example, White *et al.* (2002) suggested that disturbance of fine-grained soils during installation resulted in lower strengths, while Kirsch and Kirsch (2010) found an increase in strength in silty clay soils and a decrease in strength in sandy silt soils post installation. Further, Kirsch and Kirsch (2010) illustrated how lateral stress and strengths varied with distance from the stone column for two different soil types.

Four conclusions can be developed from this summary of case histories:

1. Stone columns have been successfully utilized on projects to increase bearing capacity, stability, and resistance to liquefaction and to decrease settlements.

2. The performance of stone columns is site-specific and varies with site conditions, load intensity, foundation flexibility, installation technique, and stone column material.
3. The installation effects of stone columns on the surrounding soil are extremely complex and at present are not well understood. But, what is understood is that installation effects do influence the performance of stone column reinforced ground.
4. A result of this extensive literature review confirms the conclusions and recommendations developed by Barksdale and Bachus (1983a) over 25 years ago, and more recently by McCabe *et al.* (2009), that there is a lack of field studies that appropriately capture all the information required to develop a complete understanding of the behavior of stone column reinforced ground.

Estimating Settlements of Stone Column Reinforced Ground

Methods for estimating settlements of stone column reinforced ground are typically based on traditional settlement analysis methods. Traditional settlement analyses are based on consolidation theory or elasticity. The following literature describes a typical design process for a stone column project, including references to conventional settlement calculations and other details unique to stone columns. After the preliminary design process is introduced, the remainder of the section describes traditional settlement analyses and design details unique to stone columns. After this introduction to traditional unreinforced settlement methods and common stone column design details, methods of estimating settlements of stone column reinforced ground are discussed.

Typical Preliminary Design Process

A typical, preliminary design process for estimating settlements of stone column reinforced ground includes the following steps (Taube and Herridge 2002):

1. Estimate the settlement for the proposed loading conditions for the unimproved ground using conventional settlement calculations.
2. Determine the reduction of settlement required to meet the design requirements. This reduction is typically expressed as a ratio of the amount of settlement of the unimproved soils to the amount of settlement of the improved soils. This ratio is often referred to as the settlement ratio or improvement factor.
3. Determine, based on experience and published empirical data, if stone columns can provide the required reduction of settlement. Determine the area replacement ratio (stone column area divided by the tributary area of the stone column) necessary to provide the required reduction of settlement.
4. Determine the stone column length, diameter and spacing required to meet the design requirements. Stone column diameter and spacing are commonly determined through design and experience. An iterative analysis is required to determine stone column length.
5. Assess the load-carrying capacity of the stone columns. The load-carrying capacity of the stone column is beyond the scope of this dissertation and will not be described in detail.

Traditional Settlement Analyses for Unreinforced Ground

Total settlement is the magnitude of downward movement of a structure or fill. Although settlement analyses have been made for hundreds of years, the estimation of settlements is still not an exact science (Coduto 1994). Settlement estimates based on laboratory consolidation tests of cohesive soils commonly range from a 100% overestimate (conservative) to a 50% underestimate (unconservative). A difficult aspect in estimating settlement is appropriately replicating the coupled soil behavior and stress distribution phenomena. Advanced modeling methods, such as the finite element method, do provide the ability to analyze each project and couple the soil behavior with the stress distribution. Such modeling is not routine in the geotechnical community at this time and is typically only conducted for special projects.

In order to estimate settlements, engineers typically determine the modulus of elasticity, or some other parameter for compressibility, such as the compression index from consolidation theory, for each new soil, and these measurements provide the parameters to utilize in settlement analyses (Coduto 1994). After this introduction to estimating settlements, methods for estimating the stress distribution within the soil are presented.

Settlement Based on Elastic Theory

The stress-deformation properties of a material in engineering mechanics can be described in terms of the modulus of elasticity, E, and Poisson's Ratio, v_p, and are defined in Equations 1 and 2, respectively (Coduto 1994). Elastic theory approaches are commonly used to estimate settlements in granular soils and heavily over-consolidated cohesive soils (Collin 2007). When a material is loaded vertically and laterally unconfined, the equations below apply. Typical values of E are provided in Table 13. Typical values of v_p are provided in Table 14.

$$E = \frac{\sigma_v}{\varepsilon_v} \tag{1}$$

$$v_p = \frac{\varepsilon_h}{\varepsilon_v} \tag{2}$$

Where: E = modulus of elasticity (also known as Young's modulus)

v_p = Poisson's ratio

σ_h = horizontal normal stress

σ_v = vertical normal stress

ε_h = horizontal normal strain

ε_v = vertical normal strain = Δh/h

Table 13. Typical values of the modulus of elasticity (after Coduto 1994)

Soil type and condition	**Modulus of elasticity, E**	
	lb/ft^2	**kPa**
	--- Undrained condition ---	
Soft clay	30,000 – 200,000	1,500 – 10,000
Medium clay	100,000 – 1,000,000	5,000 – 50,000
Stiff clay	300,000 – 1,500,000	15,000 – 75,000
	--- Drained condition ---	
Soft clay	5,000 – 30,000	250 – 1,500
Medium clay	10,000 – 70,000	500 – 3,500
Stiff clay	25,000 – 400,000	1,200 – 20,000
Loose sand	200,000 – 500,000	10,000 – 25,000
Medium dense sand	400,000 – 1,200,000	20,000 – 60,000
Dense sand	1,000,000 – 2,000,000	50,000 – 100,000

Table 14. Typical values of Poisson's Ratio

Soil Type	Poisson's Ratio, v_p
--- After Coduto (1994) ---	
Saturated soil, undrained condition	0.50
Partially saturated clay	0.30 – 0.40
Dense sand, drained condition	0.30 – 0.40
Loose sand, drained condition	0.10 – 0.30
--- After Balaam and Poulos (1983) ---	
Stone columns	0.3
Soft, normally consolidated clays	0.35 – 0.45
Medium stiff clays	0.3 – 0.35
Stiff, overconsolidated clays	0.1 – 0.3

When considering the stress-strain behavior of a soil sample in the field, the sample is subjected to some lateral confinement, as shown in Figure 6. For the condition shown in Figure 6, the idealized condition presented only permits strain in the vertical with no horizontal strain permitted such that there is a change in the height, h, but no change in the width, w. This idealized condition is known as the constrained condition (Coduto 1994). A further outcome of this idealized condition is that v_p becomes zero when no ε_h is permitted.

The constrained condition is represented in the common geotechnical laboratory test procedure known as the consolidation test. In this test, the sample is only allowed to strain in the vertical direction as the load is increased. With respect to stone columns, this idealized condition would be likely below the center of a large embankment. This constrained condition yields the constrained modulus, M, as described in Equation 3.

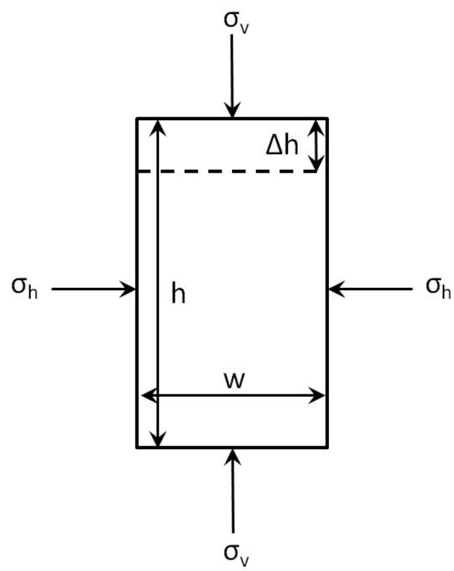

Figure 6. Idealized stress-strain condition of sample in the field

$$M = \frac{\sigma_v}{\varepsilon_v} = \frac{E(1 - v_p)}{(1 + v_p)(1 - 2v_p)} \tag{3}$$

Where: M = Constrained modulus

With E and M now defined, the settlement resulting from a change in vertical stress (σ_v) can be determined using either the modulus of elasticity or the constrained modulus, as shown in Equations 4 and 5. With regard to settlement, the resulting vertical strain is the settlement of the fill or structure based on elastic theory. However, soil is not linearly elastic and these elastic methods are approximations. The values of E and M are stress dependent, and their use results in the absence of a unique, single parameter for use in design. Even with these shortcomings, the method of elasticity remains a common procedure of estimating potential settlements. An upper bound for the settlement results when E is utilized (Barksdale and Bachus 1983a). The appropriate parameter for estimating settlements beneath the centerline of an embankment would be *M*.

Using the Modulus of Elasticity:

$$S = \frac{\Delta\sigma_v}{E} = \frac{H(\Delta\sigma)}{E} \tag{4}$$

Using the Constrained Modulus:

$$S = \frac{\Delta\sigma_v}{M} = \frac{H\ (\Delta\sigma)}{M} \tag{5}$$

Where: S = Settlement

H = Height of sample or thickness of stratum

$\Delta\sigma$ = change in vertical normal stress

The total settlement is the sum of the settlements for individual layers, such that if the modulus of elasticity is utilized, the resulting equation would be as follows:

$$S_t = \sum_{i}^{n} \frac{H_i(\Delta\sigma_i)}{E_i} \tag{6}$$

Where: S_t = Total Settlement

H_i = Thickness of sublayer *i*

$\Delta\sigma_i$ = average stress change due to pressure applied for sublayer *i*

n = number of sublayers

In practice, *E* is typically estimated empirically based on the Standard Penetration Test or from correlations with the undrained shear strength of the soil. *M* can be determined from the results of a laboratory consolidation test.

As found in textbooks such as Das (1998) and Coduto (1994), immediate settlements of fine-grained soils upon loading without any change in moisture content can be determined utilizing the theory of elasticity. However, the literature review identified no stone column case histories that included the analysis for immediate settlement and will not be discussed further. In granular soils, the routine practice is to estimate settlements utilizing *E* in the elastic theory framework.

Settlement Based on One-Dimensional Consolidation Theory

The theory of consolidation is applicable to saturated cohesive soils and has two components, primary consolidation and secondary compression. As described throughout the literature and textbooks, primary consolidation is the expulsion of water from the void spaces upon loading, and secondary compression is the adjustment of the soil fabric to the increased loading under constant effective stress after all excess pore water pressures have dissipated.

Stone columns are typically used in normally consolidated cohesive soils. Primary consolidation for normally consolidated soils can be determined utilizing the following equation:

$$S_t = \sum_{i}^{n} C_{c_i} \left(\frac{1}{1 + e_{o_i}} \right) H_i \log \left(\frac{\sigma_{vo_i} + \Delta\sigma_i}{\sigma_{vo_i}} \right) \qquad (7)$$

Where: S_t = total settlement

C_{c_i} = compression index of sublayer *i*

e_{o_i} = void ratio of sublayer *i*

H_i = Thickness of sublayer *i*

σ_{vo_i} = initial vertical effective stress at the mid-point of the compressible layer for sublayer *i*

$\Delta\sigma_i$ = increase in vertical effective stress at the mid-point of the compressible layer for sublayer *I* due to loading

n = number of sublayers

Secondary compression is an important consideration in organic and highly compressible inorganic soils (Das 1998). Secondary compression is commonly considered to result after all of the excess pore water pressures have dissipated. However, some adjustment of the soil fabric also likely occurs during the later portion of the consolidation process. The coefficient of secondary compression can be determined from a void ratio versus log time plot as follows:

$$C_\alpha = \frac{\Delta e}{\log t_2 - \log t_1} = \frac{\Delta e}{\log\left(\frac{t_2}{t_1}\right)} \tag{8}$$

Where: C_α= secondary compression index

Δe = change in void ratio from time 1 to time 2

t_1 = time 1

t_2= time 2

The magnitude of secondary compression, S_s, can be calculated using Equation 9. Stone column case histories which measured the coefficient of consolidation were shown previously in Table 9.

$$S_s = C'_\alpha \, H \, log\left(\frac{t_f}{t_p}\right) \tag{9}$$

Where: $$C'_\alpha = \frac{C_\alpha}{1+e_p} \tag{10}$$

H = thickness of layer

t_p = time at end of primary consolidation

t_f = either (a) time for completion of secondary settlement

or (b) time based on project constraints if secondary

settlement not complete.

C'_α = coefficient of secondary settlement (in terms of strain)

e_p = void ratio at end of primary consolidation

Stress Distribution

The stress distribution resulting from the applied load that changes the state of stress in both the reinforced zone and unreinforced zone is very complicated. The literature for stone column reinforced ground does not specifically address stress distribution for an embankment loading condition. The literature does address the stress distribution below footings and tanks.

An initial consideration in evaluating the stress distribution is whether the loading is rigid or flexible. Balaam and Poulos (1983) found the reduction in settlement of a flexible foundation supported by stone columns to be slightly less than that of a rigid foundation. The behavior of stone columns is quite different from an isolated stone column supporting a footing to a group of stone columns supporting a rigid footing to a large array of stone columns supporting an embankment (Wehr 2004, 2006).

Based on elastic theory, a uniformly loaded, perfectly flexible foundation bearing on an elastic material will have a sagging settlement profile, which results in the highest settlement at the center of the foundation. A rigid foundation bearing on an elastic material will exhibit uniform settlement across the entire foundation because the contact pressure will have to be redistributed due to stress distribution. Although the basic equation used in estimating elastic settlements was shown in Equation 4, an equation to determine the settlement for foundations resting on an elastic material is shown below based on stress distribution and foundation rigidity. Table 15 provides a summary of influence factors for flexible and rigid foundations (Das 1998).

$$S_e = \Delta\sigma\, B\, \frac{1 - {v_p}^2}{E}\, I_p \tag{11}$$

Where:

$$I_p = \frac{1}{\pi}\left[m_1 \, ln\left(\frac{1 + \sqrt{m_1^2 + 1}}{m_1}\right) + ln\left(m_1 + \sqrt{m_1^2 + 1}\right)\right] \tag{12}$$

S_e = elastic settlement

$\Delta\sigma$ = pressure applied upon loading

B = width of the foundation or diameter of circular foundation

I_p = nondimensional influence factor

$m_1 = \frac{length\ of\ foundation}{width\ of\ foundation}$

Table 15. Influence factors for foundations on elastic material (after Das 1998)

Shape	m_1	I_p		
		Flexible		Rigid
		Center	Corner	
Circle	- - -	1.00	0.64	0.79
Rectangle	1	1.12	0.56	0.88
	1.5	1.36	0.68	1.07
	2	1.53	0.77	1.21
	3	1.78	0.89	1.42
	5	2.10	1.05	1.70
	10	2.54	1.27	2.10
	20	2.99	1.49	2.46
	50	3.57	1.8	3.0
	100	4.01	2.0	3.43

For evaluating the stress increase in unreinforced soils due to embankment loading, Das (1995) presented two methods. The first method approximates the stress increases below the foundation based on a 2 vertical to 1 horizontal slope from the base of the applied load. This is often referred to as the 2:1 method. The second method utilizes elastic theory to determine the stress increase at any point below an embankment and provides an influence factor chart to assist in determining the influence factor. From analysis of the Priebe (1995) example calculations, a Boussinesq-type analysis was used to determine the stress increase in the untreated soils.

For an aggregate column (Geopier) technology similar to stone columns, Lawton *et al.* (1994) modified the 2:1 method for application to footings. Lawton *et al.* (1994) suggested utilizing 1.67 vertical to 1 horizontal to estimate the stress dissipation through the stone column reinforced zone. Lawton *et al.* (1994) also referenced earlier works that considered two-layered elastic strata in a Boussinesq-type analysis.

Fox and Cowell (1998) described several methods utilized to evaluate the stress distribution in the design of Geopiers. Fox and Cowell (1998) referenced work by Bowles (1982), where pressure isobars for the stress distribution below square and continuous footing using a Poisson's ratio of zero were developed by Westergaard, which is representative of the constrained condition. Fox and Cowell (1998) also presented using a traditional Boussinesq stress distribution multiplied by 0.8. Fox and Cowell (1998) also referred to the Lawton *et al.* (1994) modification to utilize a 1.67:1 stress distribution.

Sehn and Blackburn (2008) proposed a method utilizing a 4:1 stress distribution to a depth of two-thirds the length of the stone columns and a 2:1 stress distribution below this depth to determine the change in vertical stress for a footing underlain by a group of aggregate columns. Sehn and Blackburn (2008) also developed a design chart using a two-layer elastic analysis to develop stress influence factors for points below a uniformly loaded circular area.

One of the details not identified in the literature search is to what depth settlements should be determined below an embankment or structure constructed on stone column reinforced soils. Two references can be considered to provide a minimum and maximum zone of influence to consider in estimating settlements. From Sehn and Blackburn (2008), which utilized a two-layer elastic system, the zone of influence is about two times the diameter of the circular foundation, with just a little less than 10% of the stress increase remaining to be dissipated with depth. From Bowles (1982), pressure bars for a continuous footing overlying an elastic soil in a constrained condition showed that approximately 90% of the stress has been dissipated at a depth corresponding to about four times the width of the continuous footing. In the design of footings, Eurocode 7 allows the analysis to only consider the zone where the increase in effective stress is greater than 20% of the in situ effective stress (Bond and Harris 2008). The zone of influence is in the range of two to four times the width of the embankment or structure. Barksdale and Bachus (1983a) concluded that finite element analysis provided the means for estimating the vertical stress distribution beneath loadings of limited size supported by stone column reinforced ground.

Design Concepts Unique to Stone Columns

The concepts unique to the common design methods are presented in this section. The discussion of these details will be limited to defining what the terms represent and providing some basic information from the literature for reference.

Unit Cell Concept

Barksdale and Bachus (1983a) presented the unit cell idealization. For the purpose of settlement analyses, the idealization is a convenient assumption for associating the tributary area of soil surrounding each stone column with the column. Although triangular stone column spacing results in a hexagon-shaped tributary area around the stone column and a square spacing results in a square-shaped hexagonal tributary area, this area is approximated by a circular-shaped tributary area. For an equilateral triangular pattern of stone columns, the equivalent circle for the unit cell has an effective diameter based on the following:

$$D_e = 1.05\, s \qquad (13)$$

Where: D_e= effective diameter of the unit cell

s = spacing

For a square pattern, the equivalent circle has an effective diameter based on the following:

$$D_e = 1.13\, s \qquad (14)$$

The resulting cylinder of diameter, D_e, which is composed of the stone column and the tributary area around the stone column, is termed the unit cell. Barksdale and Bachus (1983a) applied the unit cell concept to an infinitely large group of stone columns subjected to a uniform loading over the large area. For purposes of analyzing settlements, a unit cell can be considered for each stone column location. Further extension of the unit cell concept applied to an infinitely large area allows the following idealizations/assumptions to be developed (Barksdale and Bachus 1983a):

1. Because of symmetry of load and geometry, lateral deformations cannot occur across the boundaries of the unit cell.
2. Also from symmetry of load and geometry, the shear stresses on the outside boundaries of the unit cell must be zero.
3. The uniform loading across the top of the unit cell must remain in the unit cell.
4. The distribution of stress within the unit cell between the stone and soil could change with depth.

Area Replacement Ratio

The area replacement ratio, $\propto_s$, is determined from stone column spacing and diameter. Stone column diameters depend on the strength of the soils being stabilized and the construction method/equipment utilized. Elias *et al.* (2006) indicated that stone column diameters vary between 0.45-1.2 m (1.5 - 4 ft) but are typically in the range of 0.9-1.1 m (3.0-3.6 ft) for the dry method, and somewhat larger for the wet method. Triangular, square, or rectangular grid patterns are used with center-to-center column spacing of 1.5 - 3.5 m (5-12 ft) (Elias *et al.* 2006).

Elias *et al.* (2006) noted that typical area replacement ratios are in the range of 0.10 to 0.40. Stone column spacing does affect performance. Bergado and Lam (1987) found that granular columns act independently at spacings of three diameters or greater.

Barksdale and Bachus (1983a) stated that the volume of soil replaced by stone columns has an important effect upon the performance of the stone column reinforced ground. The area replacement ratio quantifies the amount of soil replacement based on the following equation:

$$\propto_s = \frac{A_s}{A} \tag{15}$$

Where: $\propto_s$= area replacement ratio

A_s = area of the stone column

A = total area within the unit cell

The Priebe (1995) method utilizes the term area ratio, which is $1/\propto_s$. The area replacement ratio can also be expressed in terms of the diameter and spacing of the stone columns as follows:

$$\propto_s = C_1 \left(\frac{D}{s}\right)^2 \tag{16}$$

Where: D = diameter of compacted stone column

s = center to center spacing of stone columns

C_1 = constant dependent on pattern of stone columns

For square: $C_1 = \frac{\pi}{4}$

For triangular: $C_1 = \frac{\pi}{2\sqrt{3}}$

Stress Concentration Ratio

When an embankment is placed over stone column reinforced ground, a concentration of stress occurs in the stone column with an accompanying reduction in stress in the surrounding soil (Barksdale and Bachus 1983a). The stress concentration ratio (*SCR*) is synonymous with the terms stress ratio and stress concentration factor.

For a uniformly loaded area over evenly spaced stone column reinforced ground, the distribution of vertical stresses can be expressed as the stress concentration ratio, expressed as follows:

$$SCR = \frac{\sigma_s}{\sigma_c} \tag{17}$$

Where: SCR = stress concentration ratio

σ_s= stress on the stone column

σ_c = stress on the surrounding soil

It should be noted that many references utilize *n* to represent the stress concentration ratio (Barksdale and Bachus 1983a; Elias *et al.* 2006). However, *n* was not utilized to avoid confusion, as the Priebe method uses *n* to represent the settlement ratio.

Considering the unit cell idealization and for equilibrium of vertical forces for a given replacement ratio, the average stress, σ, over the unit cell must equal the following:

$$\sigma = \sigma_s \propto_s + \sigma_c (1 - \propto_s) \tag{18}$$

Where: σ = stress applied to the unit cell due to applied load

$\propto_s$ = area replacement ratio

For a given stress concentration ratio, the stress on the surrounding soil can be determined by the following:

$$\sigma_c = \frac{\sigma}{1 + (SCR - 1) \propto_s} \tag{19}$$

and the stress on the stone column can be determined by the following:

$$\sigma_s = \frac{SCR\ \sigma}{1 + (SCR - 1) \propto_s} \tag{20}$$

Barksdale and Bachus (1983a) stated that the above two equations, which give the stress due to the applied loading in the stone column and surrounding soil, are extremely useful in settlement analyses. However, a reasonable value of the stress concentration ratio must be determined. Elias *et al.* (2006) indicated that a high *SCR* (3 to 4) may be warranted if the in situ soil is very weak and the column spacing small. For stronger soils and wider spacing, lower stress ratios have been indicated in the range of 2 to 2.5. For preliminary design, a ratio of 2.5 is often conservatively utilized (Elias *et al.* 2006). For additional guidance on selection of a *SCR*, other reported values for the stress concentration ratio are presented in Table 16.

Table 16. Summary of *SCR* findings or recommendations

Reference	*SCR* findings or recommendations
Goughnour and Bayuk 1979b	For a load test on 45 stone columns at s = 5.8 ft on a triangular, D = 4 ft, L = 20.5 ft, $\propto_s$=0.43, an initial *SCR* of 3.0 was observed to decrease with time to 2.6.
Barksdale and Bachus 1983a	For an embankment with stone columns at s = 5.7 ft on a square grid, D = 3 ft, L = 22-26 ft, $\propto_s$=0.25, an average *SCR* of 2.8 was observed and found to be approximately constant with time.
	For embankments supported with sand compaction piles with $\propto_s$ from 0.1 to 0.3 and variable lengths, *SCR* was observed to range from 2.5-8.5 with an average 4.9 and was observed to increase with time.
	For a model test with $\propto_s$ from 0.07 to 0.4 and variable lengths, *SCR* was found to range from 1.5-5.
Sarkar *et al.* 1983, Munfakh *et al.* 1984	At the Jourdan Road Terminal, found the *SCR* to be between 2.5 and 3.5 at the end of construction and between 4 and 5 at the end of consolidation.
Mitchell and Huber 1985	Found the *SCR* in the range from 2 to 6, with values of 3 to 4 typical.
Sheng 1986	Measured *SCR* from seven projects in China ranged from 1.5 to 3.5.
Bergado *et al.* 1987	*SCR* of 2 after long-term monitoring of embankment loading, much higher *SCR* shown for short term, rigid plate load tests
Allen *et al.* 1991	Measured *SCR* for a cut and cover tunnel project less than 1 during construction and 1 after construction completed.
Greenwood 1991	For widespread loading on columns in soil in which excess pore pressures are insignificant, the stress concentration ratio progressively reduces. *SCR* of about 4 to 6 at working loads are observed, similar to the principal stress ratio in the column, which implies a vertical to horizontal stress ratio in the soft surrounding clay soils close to 1.
Stewart and Fahey 1994	Centrifuge testing resulted in *SCR* from about 2 to 3.5.
Ashmawy *et al.* 2000	*SCR* between 2.5 and 4.5 should be used in conjunction with analytical methods, which is consistent with current practice.
Samieh 2002	In a numerical study, the *SCR* was found to be 2.6 near the embankment centerline and around 3.1 at the toe of the embankment.
Pitt *et al.* 2003	Measured *SCR* of approximately 3, 5, and 6.

Table 16. (continued)

Reference	***SCR* findings or recommendations**
Liu *et al.* 2009	Study indicated *SCR* for columns constructed with crushed stone to be 4.
Stuedlein 2010	An average and coefficient of variation of SCR of 3.3 and 40% appears warranted for use in preliminary designs for aggregate piers. This is based on stone columns, aggregate piers, and sand compaction piles.

Barksdale and Bachus (1983a) developed a design chart from finite element modeling that allows determination of the stress concentration ratio based on the length of the stone column, *L*; the diameter of the stone column, *D*; the modulus of elasticity of the clay; and considering an area replacement ratio of 0.25. Soil mechanics and most case histories, such as Han and Ye (2001), indicate that the *SCR* will increase with time as settlements occur in the surrounding soil, allowing the stone column to carry more load. However, some case histories have found the *SCR* to decrease with time, such as Goughnour and Bayuk (1979b).

Elias *et al.* (2006) discussed a number of variables that affect the *SCR*, such as the relative stiffness of the stone column and the soil, the length of the stone column, area ratio, and any granular materials placed over the stone columns. Table 17 provides a summary of many factors that influence the stress concentration ratio. In estimating settlements of stone column reinforced ground using the Equilibrium method, a proper determination of the *SCR* is critical. From the many factors described in Table 17, the determination of the *SCR* is complex and often based on experience with previous projects. Numerical methods can be used to estimate the stress distribution between the stone columns and surrounding soil (Kirsch and Sondermann 2003).

Table 17. Summary of factors influencing *SCR*

Factor	Comment
Diameter of stone column	Through geometry, the diameter of stone column controls strength and compressibility properties of the unit cell.
Length of stone column	The length of the stone column influences the failure type, such as bulging or base (toe) plunging, and the failure type indicates the downward movement properties of a stone column.
Spacing of stone columns	The spacing of stone columns affects the interaction between stone columns and the area of the unit cell.
Shear strength and compressibility of soil	The shear strength and compressibility of the soil influence the stone column shape and diameter, as well as the bulging characteristics upon loading.
Shear strength and compressibility of stone column	The type of material used for construction of the stone column, such as either rounded or angular aggregate, will affect the shear strength and compressibility of the stone column. The stiffness ratio of the stone column to the surrounding soil is consideration.
Loading platform above the stone columns	The stiffness or flexibility of the loading platform affects the spreading or distribution of the loading stresses across the stone columns and soil.
Load intensity/Rate of load	The intensity and rate of loading affect the settlement characteristics of the soil and stone column.

Settlement Ratio or Improvement Factor

The literature is mixed on the use of the terms settlement improvement ratio and improvement factor. Barksdale and Bachus (1983a) defined the settlement ratio as the settlement of the reinforced ground to that of the unreinforced ground, which results in a value less than 1. Elias *et al.* (2006) defined the settlement ratio as the settlement of unreinforced ground to that of reinforced ground, which results in a value greater than 1. The Priebe (1995) method utilizes the improvement factor, n, which is defined as the settlement of unreinforced ground to that of reinforced ground.

Regardless of terminology, the ratios are intended to quantify the improvement (reduction) in settlement of stone column reinforced ground compared to the same ground untreated. Taube and Herridge (2002) reported settlement ratios between 2 and 3 (i.e., settlement can be reduced by a factor of between 2 and 3).

Equal Strain Assumption

An important underlying assumption for many methods of estimating settlements is commonly referred to as the equal strain assumption. This assumption considers that the deflection of both the stone column and the surrounding soil in the unit cell upon loading are approximately the same (Barksdale and Bachus 1983a; Priebe 1995; Xie *et al.* 2009). Early studies, such as Goughnour and Bayuk (1979b), yielded field measurements to support this assumption. Elias *et al.* (2006) indicated that both field measurements and finite element analyses have indicated this assumption to be valid. However, Ashmawy *et al.* (2000) and White *et al.* (2002) completed studies that yielded measurements that question the validity of this assumption.

Estimating Settlement of Stone Column Reinforced Ground

The focus of this study is on the settlement of stone column reinforced ground. Greenwood (1991) concluded that under widespread vertical loads, ground strengthened by arrays of columns behave in complex ways. Early methods of estimating settlements of stone column reinforced ground were strictly empirical and semi-empirical. Theoretical models of the inter-relationship between the stone columns and the in situ soil were presented in the 1970s. Since the 1970s, over 16 design methods have been reported to estimate the settlement of stone column reinforced soil. The design methods developed have been based on theory alone, limited field data, a combination of theory and field data, laboratory experiments, and/or modeling studies. Two approaches to estimating settlements were found in FHWA manuals:

1. Barksdale and Bachus (1983a) in *Design and Construction of Stone Columns* recommend utilizing the Equilibrium method for the upper bound and the nonlinear finite element model (FEM) Settlement Charts method for the lower bound. The best estimated settlement should be taken as the average of the two calculations. For settlement calculations using the Equilibrium method, a *SCR* of 4.0 to 5.0 was recommended.
2. The current recommended design procedure for preliminary estimates presented by Elias *et al.* (2006) in *Ground Improvement* is to utilize the Priebe method to evaluate the upper bound settlement and cost at various spacings. The Equilibrium method with a *SCR* of 3 is then utilized to determine the lower bound of effectiveness. Elias *et al.* (2006) does not explicitly address averaging results of two methods for estimating settlements.

A brief listing and characteristics of the methods identified are presented in Table 18. Brief introductions, in chronological order, to the methods follow after the table. Extended introductions to the Equilibrium and Priebe methods are provided, as those methods are evaluated in Chapters 5 and 6. Other considerations during stone column design that are not described in detail include time rate of settlement, bearing capacity, shear strength increase, and seismic.

Table 18. Summary of methods for estimating settlements

Method	Method details: Unit cell idealization (Yes/No)	Equal strain assumption (Yes/No)	Method theory	Untreated settlement required (Yes/No)	Comments
Greenwood (1970)	No	No	No	Yes	Empirical correlation with spacing of columns and strength of clay soils.
Hughes and Withers (1974)	Yes	Yes	Plastic	Yes	Early design method for widespread loading.
Incremental method (Goughnour and Bayuk 1979a)	Yes	Yes	Elastic-Plastic	Yes	Considered load intensity in elastic-plastic behavior.
Balaam and Booker (1981 and 1985)	Yes	Yes	Elastic	Yes	Results similar to Priebe method. Considered rigid foundation.
Balaam and Poulos (1983)	Yes	Yes	Elastic-Plastic	Yes	Results similar to Priebe method. Both rigid and flexible loading.
Equilibrium (Barksdale and Bachus 1983a)	Yes	Yes	None	No	Uses the SCR to determine stress reduction in soil to estimate settlements.
FEM Settlement Charts (Barksdale and Bachus 1983a)	Yes	Yes	Elastic-Plastic	No (requires column length)	Incorporates load dependent behavior of overall system.
Van Impe and De Beer (1983)	No Plane Strain	Yes	Elastic	Yes	Design charts to estimate settlements.
Greenwood and Thomson (1984)	Design chart based on the Priebe method.				Included because cited in case history

Table 18. (continued)

Method	Method details: Unit cell idealization (Yes/No)	Equal strain assumption (Yes/No)	Method theory	Untreated settlement required (Yes/No)	Comments
Priebe (1995)	Yes	Yes	Elastic	Yes	Considered infinitely wide reinforced area originally, modified for footings in 1995.
Chow (1996)	Yes	Yes	Yes	Yes	Simple method developed for sand compaction piles. Similar results to the Balaam and Booker.
Alamgir *et al.* (1996)	Yes	No	Elastic-Plastic	Yes	Allowed surrounding soil to settle more than stone column.
Poorooshasb and Meyerhof (1997)	Yes	Yes	Elastic-Plastic	Yes	Priebe method is special case of general equation derived for study.
Pulko and Majes (2005)	Yes	Yes	Elastic-Plastic	Yes	Considered rigid footings.
Ambily and Gandhi (2007)	Yes	Yes	Elastic-Plastic	Yes	Similar results to the Priebe method.
Borges *et al.* (2009)	Yes	Yes	Elastic-Plastic	Yes	Results in the range of Priebe method, and Balaam and Booker

No standard terminology exists for stone column design, and some methods use the same terms or symbols with different meanings.

Greenwood Method (1970)

Greenwood (1970) presented a chart and noted the estimation of settlement was still empirical due to the unavailable, rigorous solutions to ensure soil and stone column compatibility. Greenwood (1970) presented an empirical method to estimate the settlement, "as precise mathematical solutions ensuring compatibility of column and clay deflections have not been derived." The curves were based on case histories for widespread loadings as a function of column spacing and shear strength. A range of settlement improvement was shown for clay strengths between 20 and 40 kPa (400 and 800 psf) for vibro-replacement. A single curve for a clay strength of 40 kPa (800 psf) was shown for vibro-displacement.

The Greenwood method appears to give reasonable results for an undrained shear strength of 400 psf and area replacement ratios less than about 0.15 (Barksdale and Bachus 1983a). For firm soils and usual levels of ground improvement, Barksdale and Bachus (1983a) suggested that the improvement factors from Greenwood's method appear to be high.

Hughes and Withers (1974)

Hughes and Withers (1974) used plastic theory to present a method of analyzing both bearing capacity and settlement. The case of a widespread loading was considered, and the load dependent behavior of stone columns was emphasized. Equal strain of the stone column and surrounding soil was utilized to estimate the benefit of stone columns in reducing the settlement. Further, Hughes and Withers (1974) stressed that an allowance must be made for compressible soil below the stone column reinforced zone, which could possibly dominate the total settlements.

Incremental Method (Goughnour and Bayuk 1979a)

Goughnour and Bayuk (1979a) developed an analysis method that extended the earlier empirical and semi-empirical design methods of Hughes *et al.* (1975), Baumann and Bauer (1974), and Priebe (1976). Goughnour and Bayuk (1979a) presented an analysis that included compressibility of the in situ soil, plastic and elastic behavior of the stone, stress distribution between the stone columns and the in situ soil, and time-settlement relationships for the composite mass. Goughnour and Bayuk (1979a) provided the theoretical framework for the Incremental method, and a well-documented case history was provided as a successful application of the design method in Goughnour and Bayuk (1979b).

Goughnour (1983) developed design charts to assist in hand calculations using the Incremental method. The Incremental method is similar to other methods that require calculation of settlement without stone columns and then provide a method for estimating the reduction in settlement due to inclusion of the stone columns. Glover (1985) indicated that the procedure developed by Goughnour and Bayuk (1979a) provided an advance in the design of stone columns.

Balaam and Booker Method (1981 and 1985)

Balaam and Booker (1981) developed a method utilizing elastic methods to determine the magnitude and rate of settlement for rigid foundation supported by granular pile reinforced ground. The unit cell was utilized in analyzing soil stresses and strains assuming no yielding of the clay or soil. The shearing stresses and moments in the rigid mat were also analyzed. Balaam and Booker (1985) extended their earlier work to consider yielding of the column. A comparison completed by Balaam and Booker (1985) found that the Balaam and Booker method results are comparable to those of the Priebe (1976) method.

Balaam and Poulos (1983)

Balaam and Poulos (1983) utilized the finite element method to reproduce the response of previously published field load data. The stone column and clay were both treated as elastic, perfectly plastic materials obeying a Mohr-Coulomb yield criterion. The behavior of both rigid foundations and uniformly loaded flexible foundations were considered. A settlement comparison by Balaam and Poulos (1983) was found to yield similar results to those of the Priebe (1976) method.

Equilibrium Method

The Equilibrium method is a simple procedure for estimating the settlement of stone column reinforced ground. In using this approach, the stress concentration factor must be estimated using either experience or the results of field stress measurements, such as those obtained from full-scale embankments. Lower estimates of stress concentration factors result in more conservative (larger) settlement predictions. The Equilibrium method requires the following assumptions:

- The unit cell idealization is valid.
- The total vertical load applied to the unit cell equals the sum of the force carried by the stone and the surrounding soil (i.e., equilibrium is maintained within the unit cell).
- The vertical displacements of stone column and the surrounding soil are equal.

- A uniform vertical stress due to external loading exists throughout the length of the stone column.

The Equilibrium method includes the following steps:

1. Estimate a value to be used as the stress concentration ratio (*SCR)*. The *SCR* is the ratio of vertical stress in the stone column to the vertical stress in the surrounding soil.
2. Calculate the area replacement ratio for the design. The area replacement ratio, $\propto_s$, is equal to the area of the stone column divided by the total area of the unit cell.
3. Determine the resulting final vertical stress in the surrounding soil, σ_c.
4. Use conventional one-dimensional consolidation theory to estimate the settlement of the stone column improved soil assuming compression under the estimated vertical stress in the matrix soil.

This design method indicates that longer stone columns and smaller applied stresses result in a greater settlement reduction (Barksdale and Bachus 1983a). When using the Equilibrium method, settlements occurring beneath the reinforced ground must be considered separately using conventional consolidation or elastic settlement analyses. Example settlement calculations utilizing the Equilibrium method are provided in Barksdale and Bachus (1983b) and Barksdale (1987).

FEM Settlement Charts Method

According to Barksdale and Bachus (1983a), the finite element method offers the most theoretically sound approach for modeling stone column improved ground. Barksdale and Bachus (1983a) provide charts for predicting the settlement of stone column reinforced sands and silty sands, which were developed using linear elastic theory. This case considered ${}^{E_s}/_{E_c} \leq 10$, where E_s and E_c are the average modulus of elasticity for the stone column and soil, respectively. The Poisson's ratios of the stone and matrix soil were assumed to be 0.30 and 0.35, respectively. The linear elastic settlement influence factor charts are provided in Barksdale and Bachus (1983a). Example settlement calculations utilizing the FEM Settlement Charts are provided in Barksdale and Bachus (1983b), and Barksdale (1987). Barksdale and Bachus (1983a) provide settlement charts for stone columns in cohesive soils that considered $E_s/E_C \geq 10$, where E_s is modulus of the stone column and E_c is the modulus of the surrounding granular soils. The stress concentration ratio, length to diameter ratio, and the elasticity of the stone column to the soil ratio are the three inputs in this procedure. The curves were developed for a representative stone column angle of internal friction of 42 degrees and a coefficient of at-rest earth pressure of 0.75 for both the stone column and the soil. The clay was modeled as an elastic-plastic material, and the stone was taken to be stress-dependent. The nonlinear settlement charts are presented in Barksdale and Bachus (1983a). The charts use the average applied stress, the modulus of the clay, and the modulus of the soft boundary (in situ soil) to determine the average vertical strain.

Van Impe and De Beer (1983)

Van Impe and De Beer (1983) considered two cases in which (1) under the foundation load the columns are at the limit of equilibrium and deform at constant volume, and (2) under the foundation load the stone columns deform elastically. In consideration of the constant volume approach, the problem was simplified into an elastic, plane strain condition. A series of design charts were presented that indicate the vertical settlement of the composite layer divided by the vertical settlement of the natural soft layer. In consideration of the second case for stone columns deforming elastically, the modulus of elasticity and Poisson's ratio were introduced. In reducing the equations for the second case, Van Impe and De Beer (1983) concluded that the requirements for a linear elastic analysis are generally not met in stone column practice. Van Impe and De Beer (1983) recommended using the computational methods for case 1 and broadly state that the computational method has been found to be reliable when applied to some foundation problems of large storage tanks on soft soil improved with stone columns.

Greenwood and Thomson (1984)

A case history reported by Maduro *et al.* (2004) referenced a design method by Greenwood and Thomson (1984). Greenwood and Thomson (1984) authored an Institution of Civil Engineers (ICE) Works Construction Guidelines document titled *Ground Stabilization: Deep Compaction and Grouting*. Greenwood and Thomson (1984) provide an illustration based on the Priebe (1976) method and reference Greenwood (1970) where a preliminary approximation for isolated shallow footings is that settlements will be reduced by around 50% by utilizing stone columns.

The Priebe Method

Priebe (1995) provides a design procedure for vibro-replacement construction of stone columns. Priebe initially published the design procedure in 1976 in German. Since the initial work in 1976, Priebe adapted, extended, and supplemented the design procedure as found in Priebe (1991), and Priebe's work culminated in the procedure set forth in Priebe

(1995). Priebe (1995) provides design procedures and design charts for various aspects of stone column design, including settlement reduction, bearing capacity, shear values of improved ground, settlement of footings, and liquefaction. The procedures associated with analyzing the reduction in settlements are summarized below. Greenwood and Kirsch (1984) concluded that the simplicity of the Priebe method applying an improvement ratio to conventional consolidation is attractive to engineers, which results in the method being widely used. The Priebe method is a common method for design in industry. For example, Chambosse and Dobson (undated-a) with GKN Keller and The Vibroflotation Group utilize the Priebe method in determining settlements. Although not detailed in this summary, Priebe (1995) provides a method for estimating settlements of footings.

Priebe (1995) contrasts vibro-replacement with vibro compaction and concludes that only considerable efforts like large-scale load tests can prove the benefit of stone columns. The Priebe method quantifies the improvement that results from the inclusion of the stone column without any quantification of the densification of the soil between stone columns. Or as alternatively stated by Priebe (1995), “The design method refers to the improving effect of stone column in a soil which is otherwise unaltered in comparison to the initial state.” If the installation changes the engineering properties of the soil between the columns, the soil must be evaluated before the design of vibro-replacement can be accomplished.

The complex system of vibro-replacement allows a more or less accurate evaluation only for the case of an infinite loading area over an infinite column grid. The Priebe method determination of the improvement factor makes the following assumptions:

1. The column is bearing on a rigid layer.
2. The stone is incompressible.

3. The bulk densities of the stone and soil are neglected.

4. Any settlement of the loaded are is due to the bulging of the column, which is constant over the length of the column.

5. During construction, the soil is displaced so that the coefficient of lateral earth pressure is equal to one.

An equation is provided below for predicting the improvement factor based on the cross-sectional area of the column, the area of the unit cell, and the coefficient of active earth pressure. The series of equations used to develop the basic improvement factor, n_o, consider the coefficient of earth pressure to be one and are presented below.

$$n_o = 1 + \frac{A_c}{A}\left[\frac{1/2 + f\left(\mu_s, A_c/A\right)}{K_{ac}\ f\left(\mu_s, A_c/A\right)} - 1\right] \quad (21)$$

Where:

$$f\left(\mu_s, A_c/A\right) = \frac{(1-\mu_s)\left(1 - A_c/A\right)}{1 - 2\mu_s + A_c/A} \quad (22)$$

$$K_{ac} = tan^2\left(45° - \phi_c/2\right) \quad (23)$$

n_o = basic improvement factor
A_c = area of column
A = unit cell area
μ_s = Poisson's ratio
K_{ac} = Rankine's active earth pressure
ϕ_c = stone column material friction angle

Utilizing a Poisson's ratio of 1/3, which Pribe (1995) suggested, leads to the following expression:

$$n_o = 1 + \frac{A_c}{A}\left[\frac{5 - {A_c}/{A}}{4\,K_{ac}\ f\left(1 - {A_c}/{A}\right)} - 1\right] \tag{24}$$

The basic improvement factor resulting from the above equation for various friction angles of stone column material are shown by Priebe through a set of design curves. Note that for the Priebe method, A_c is the area of the column, and the area ratio (A/A_c) is the inverse of the area replacement ratio. Priebe (1995) extended the design procedure to consider column compressibility, overburden, and compatibility controls.

Consideration of Stone Column Compressibility. The stone in stone columns exhibits some compressibility under an applied load, which is not considered in the basic improvement factor. To account for this, Priebe (1995) developed an approach to predict an addition to the area ratio based on the constrained modulus ratio of the soil to the stone column and the friction angle of the stone column. This addition to the area ratio can then be added to the computed area ratio. The resulting area ratio can then used to determine a reduced improvement factor, n_1, which accounts for stone column compressibility. The consideration of stone column compressibility will result in more settlement.

Consideration of the Overburden. The overburden pressure in the soil increases with depth, which corresponds to an increase in stone column capacity due to the increased lateral support provided by the soil surrounding the column. The external load should not be included in this consideration. Priebe (1995) accounts for this through a depth factor, f_d, which results in a value greater than 1. The final improvement factor, n_2, is equal to $f_d \cdot n_1$. The consideration of the overburden results in less settlement.

Compatibility Controls. The considerations for stone column compressibility and overburden are independent of each other. Priebe (1995) developed a compatibility control to ensure that no more load is assigned to the columns that they can bear with respect to their compressibility.

The first compatibility control is the depth factor, f_d, which limits the load assigned to the column such that the compressibility of the stone column resulting from the applied load does not exceed the settlement of the composite system. A depth factor less than 1 should not be considered, even though it is mathematically possible. This first control applies when the surrounding soil is fairly dense or stiff.

The second compatibility control limits the maximum value of the improvement factor. This control is similar to the first in that it limits the settlement of the stone columns based on the settlement of the surrounding soil from the applied load. This second control applies when loose or soft soils are encountered and described by the following equation:

$$n_{max} = 1 + \frac{A_c}{A}\left(\frac{D_C}{D_S} - 1\right) \quad (25)$$

Where: n_{max} = maximum improvement factor

A_c = area of column

A = unit cell area

D_C = constrained modulus stone column

D_S = constrained modulus soil

Proportional Load on Stone Columns. The stone columns are stiffer than the surrounding soil, which results in more of the load being attracted to the columns than the soil. Priebe (1995) refers to this stress concentration as the proportional load on the stone columns. The friction angle of the stone, the area ratio, and the improvement factor are used as inputs.

The design procedure presented by Priebe (1995) does not consider the volume decrease in the surrounding soil resulting from the bulging of the columns under the applied load. This volume decrease in the soil results in more load being carried by the soil than actually calculated, making a reduction of the proportional load factor necessary. Priebe (1995) concluded that his method "seems to be adequate."

Chow (1996)

A solution for the settlement of subsoil improved with sand compaction piles was developed by Chow (1996). Chow (1996) named the solution the Simplified method. The Simplified method uses elastic theory and assumes the unit cell deforms under one-dimensional confined compression. Chow (1996) developed a solution that resulted in identical expressions as compared with Aboshi *et al.* (1979). Chow (1996) found the Simplified method to compare well with the theoretically rigorous method of Balaam and Booker (1981).

Alamgir, Miura, Poorooshasb, and Madhav (1996)

Alamgir *et al.* (1996) developed a solution that allows the soil between the stone columns to deflect more than the stone columns at the surface. Further, Alamgir *et al.* (1996) allowed a gradual transfer of vertical stresses from the stone column to the surrounding soil with increasing depth. The method developed by Alamgir *et al.* (1996) was shown to compare with the results of a finite element analysis. A key finding of Alamgir *et al.* (1996) was that the relative stiffness of the column and soil has a significant effect on the reduction of settlement and that the Poisson's ratio has little effect on the reduction of settlement.

Poorooshasb and Meyerhof (1997)

Poorooshasb and Meyerhof (1997) analyzed end bearing stone columns and lime columns. The analysis considered a large number of regularly spaced stone columns of equal length installed in a weak soil layer. A rigid mat was considered over the stone column reinforced ground. Poorooshasb and Meyerhof (1997) developed a chart solution to determine the settlement reduction.

Pulko and Majes (2005)

Pulko and Majes (2005) developed a "simple and accurate prediction of settlements of stone column reinforced soils." The unit cell assumption was utilized, and a rigid footing was taken to bear over stone column reinforced ground. The prediction method results in a settlement reduction factor that is applied to the calculated untreated settlement. The prediction method was found to compare favorably with the results of a finite element analysis completed by Pulko and Majes (2005).

Ambily and Gandhi (2007)

Ambily and Gandhi (2007) completed a laboratory experiment followed by a finite analysis. Small groups of columns were evaluated at various spacings. Ambily and Gandhi (2007) found that columns at a spacing greater than three times the column diameter do not yield any significant improvement. Ambily and Gandhi (2007) compared their work with Balaam *et al.* (1977) and Priebe (1995), and found the solution very similar to the Priebe (1995) method.

Borges, Domingues, and Cardoso (2009)

Borges *et al.* (2009) developed a new design method relating the area replacement ratio to an improvement factor based on the results of finite element modeling. Although the correlation is similar to Priebe (1995), the ratio of the compression index of the soil to the column material is utilized instead of the stone column material angle of internal friction. An example of the proposed design method presented by Borges *et al.* (2009) was found to be in the range of estimates from the Balaam and Booker (1985) method and the Priebe (1995) method.

Existing Software

During the literature review, references were found for existing software that can analyze settlements of stone column reinforced ground. These programs are provided to highlight the methods commonly available in commercial programs.

Columns 1.01 is a program developed by M. Bouassida and L. Hazzar that has a coupled approach to designing for both bearing capacity and settlements (Bouassida *et al.* 2009b). The program has been developed for rigid loading conditions. The program allows the user to evaluate settlements using the following methods: Balaam and Booker (1981), Chow method (1996), Variational method proposed by Bouassida *et al.* (2003) (*in French*), and Normes Françaises NFP 11-212 (2005) (*in French* - French recommendations).

GRETA is a commercial program developed in cooperation with Mr. Priebe. The program can analyze settlements for single footings, strip footings, and uniform loadings and is based on the Priebe (1995) method.

DC-Vibro is a commercial program developed utilizing the Priebe method (1995). Program information indicates DC-Vibro was developed in cooperation with the Vibroflotation Group.

StoneC v.4.0 is a commercial program developed utilizing the Priebe method (1995). Program information indicates that StoneC was tested by the Vibroflotation Group and performed well.

Other Required Analyses in Addition to Estimating Settlements

The literature review resulted in the identification of methods for design procedures in addition to settlement analysis. This section provides a partial listing of the methods identified during the review.

Most settlement analyses of cohesive soils also consider the time required for consolidation. The following references were identified through this study, which can be of benefit in analyzing the consolidation rate: Barksdale and Bachus (1983a), Han and Ye (2001, 2002), De Silva (2005), Fessi and Bouassida (2005), Zhang *et al.* (2006), Andreou *et al.* (2008), Castro and Sagaseta (2009a, 2009b), Cimentada and Da Costa (2009), Wang (2009), Han (2010), Kirsch and Kirsch (2010), and Cimentada *et al.* (2011).

Other analyses must consider bulging failure, column failure at the base, and general shear failure in design. The bearing capacity of the treated ground is three to four times greater than that of the untreated ground (Bergado *et al.* 1984; Bergado and Lam 1987). The following references were identified through this study, which can be of benefit in analyzing the bearing capacity of stone columns: Greenwood (1970), Hughes and Withers (1974), Brauns (1978), Barksdale and Bachus (1983a), Guo and Qian (1991), Bouassida *et al.* (1995), Van Impe *et al.* (1997b), Jellali *et al.* (2005), Elias *et al.* (2006), Etezad *et al.* (2006), Zhang *et al.* (2009), Kirsch and Kirsch (2010), and Chambosse and Dobson (undated-b).

Barksdale and Bachus (1983a, 1983b) provide three analysis methods for slope stabilization: the Profile method, the Composite Shear Strength method, and the Lumped Moment method. Other approaches were presented by Meade and Allen (1985), Christoulas *et al.* (1997), and Chambosse and Dobson (undated-c).

Liquefaction mitigation due to densification is discussed in Baez and Martin (1993, 1995), Mitchell *et al.* (1995, 1998), Priebe (1995), Rizzo *et al.* (1997), Goughnour and Pestana (1998), Blewett and Woodward (2001), Adalier *et al.* (2003), Adalier and Elgamal (2004), Elias *et al.* (2006), Shenthan *et al.* (2006), Noorzad *et al.* (2007), Olgun and Martin (2008), Rollins *et al.* (2009), and Kirsch and Kirsch (2010).

The working platform often also serves as a load transfer platform, as described in Osbaldeston and Phear (2000). References discussing the load transfer platform aspect include Ambily and Gandhi (2006), Filz and Smith (2006), Abdulah and Edil (2007a, 2007b), Deb *et al.* (2007, 2008, 2010), Deb (2008, 2010), Huang *et al.* (2009), and Deb and Dhar (2011).

Observations of Settlement Estimating Methods

In reviewing the current analytical methods, the following observations can be developed:

- Each method, except for the Equilibrium method and the FEM Settlement Charts method, results in a ratio to estimate stone column treated ground settlement based on the estimated untreated settlements.
- The complete response of the stone column reinforced ground is only determined in the Incremental method and the FEM Settlement charts method.
- A limitation of all but one of the analytical methods identified is the general assumption that the unit cell undergoes uniform deformation over both the stone and the soil.
- The installation effects are not considered in any of the identified methods. Priebe (1995) alludes to post-installation testing to identify the appropriate soil parameters for settlement estimates.
- Small variations in the stress concentration ratio can result in very large differences in the relative distribution of load supported by the column and the soil, which ultimately results in a large scatter of estimated settlements for a specific project. (Ashmawy *et al.* 2000).
- Numerical and analytical models are of limited value for settlement prediction due to the difficulty of obtaining accurate soil and stone properties. Numerical modeling does offer the benefit of providing information regarding distribution of stresses and strains, as well as mechanism of stress transfer between the soil and the stone column (Ashmawy *et al.* 2000).

CHAPTER 3. WEB-BASED INFORMATION SYSTEM FOR GEOCONSTRUCTION TECHNOLOGIES IN TRANSPORTATION INFRASTRUCTURE

Modified from a paper submitted to the *Journal of Computing in Civil Engineering*, published by the American Society of Civil Engineers (ASCE)

S. Caleb Douglas[1 2], P.E., M. ASCE, Vernon R. Schaefer[3 4 5], P.E., M. ASCE,

Ryan R. Berg4[6], P.E., D.GE, F. ASCE, and Peter Hunsinger4[7]

Abstract

The geotechnical engineering community currently lacks a central repository that summarizes, distills, and distributes the abundant amount of information regarding geoconstruction technologies. A new comprehensive, web-based information system compiles this knowledge for 46 geoconstruction technologies applicable to transportation infrastructure in the following areas: ground improvement, geosynthetics, grouting, slope

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[1] Ph.D. candidate, Civil Engineering, Iowa State University, Ames, IA 50010-3232. E-mail: calebd@iastate.edu
[2] Primary researcher and author
[3] Professor of Civil Engineering, Iowa State University, Ames, IA 50010-3232. E-mail: vern@iastate.edu
[4] Secondary author
[5] Author for correspondence
[6] Ryan R. Berg & Associates, Inc., Woodbury, MN, 55125-3504. E-mail: ryanberg@att.net
[7] Web Manager, Institute for Transportation, Iowa State University, Ames, IA 50010-8664. E-mail: rpeterh@iastate.edu

stabilization, soil reinforcement, soil stabilization, alternative materials, and recycling. The information system contains an introduction to the Geotechnical Design Process, Glossary, Catalog of Technologies, and Technology Selection assistance. For each technology, the following documents can be accessed through the Catalog of Technologies: Technology Fact Sheet, Photographs, Case Histories, Design Guidance, Quality Control/Quality Assurance, Cost Information, Specifications, and Bibliography. Technology selection assistance aids the user in identifying potential geoconstruction technologies for a user-defined set of project conditions. The target audience for the system is primarily public agency geotechnical engineering personnel, transportation managers, and decision makers at local, state, and federal levels. However, civil/structural, construction, pavement, and construction engineers in consulting, contracting, and academia will also find the system useful.

Introduction

Technologies used in geotechnical design and construction have developed markedly over the past five decades. Although many geoconstruction technologies are commonly utilized in various areas of the U.S., other geoconstruction technologies face both technical and non-technical obstacles preventing broader utilization. A web-based information system was developed to overcome these obstacles for the forty-six geotechnical materials, systems, and technologies (referred to as 'geoconstruction technologies') shown in Table 19. A user will be able to assess and implement each technology from the information provided in the system.

Comprehensive, automated systems for geoconstruction technologies have been developed in the past, but the systems are either currently unsupported or publicly unavailable (Chameau and Santamarina 1989; Motamed *et al.* 1991; Sadek and Khoury 2000). The value of this web-based information system for geoconstruction technologies is that it collects, synthesizes, integrates, and organizes a vast amount of critically important information about geotechnical solutions in a system that makes the information readily accessible to the user.

Table 19. Geoconstruction technologies in the information system

Geoconstruction Technologies	
Aggregate columns	Geotextile encased columns
Beneficial reuse of waste materials	High-energy impact rollers
Bio-treatment for subgrade stabilization	Hydraulic fill + vacuum consolidation + geocomposite drains
Blasting densification	Injected lightweight foam fill
Bulk-infill grouting	Intelligent compaction
Chemical grouting/injection systems	Jet grouting
Chemical stabilization of subgrades and bases	Lightweight fill, eps geofoam, low-density cementitious fill
Column-supported embankments	Mechanical stabilization of subgrades and bases
Combined soil stabilization with vertical columns	Micro-piles
Compaction grouting	Mechanically stabilized earth wall systems
Continuous flight auger piles	Onsite use of recycled pavement materials
Deep dynamic compaction	Partial encapsulation
Deep mixing methods	Prefabricated vertical drains (PVDs) and fill preloading
Drilled/grouted and hollow bar soil nailing	Rapid impact compaction
Electro-osmosis	Reinforced soil slopes
Excavation and replacement	Sand compaction piles
Fiber reinforcement in pavement systems	Shoot-in soil nailing
Geocell confinement in pavement systems	Screw-in soil nailing
Geosynthetic reinforced construction platforms	Shored mechanically stabilized earth wall system
Geosynthetic reinforced embankments	Stone columns
Geosynthetics reinforcement in pavement systems	Vacuum preloading with and without PVDs
Geosynthetics separation in pavement systems	Vibrocompaction
Geosynthetics in pavement drainage	Vibro-concrete columns

The web-based information system is best described as a decision support system. The web-based system will be utilized by both technical and nontechnical personnel for all types of transportation-related projects. Each project will have a unique set of varying field, loading, and boundary conditions. A decision support system shifts the role of computers from one of generating data and information to a more advanced function of supporting (in a variety of ways) decision making in complex and ill-structured task settings (Hopple 1998). Experienced engineers will benefit in decision making from the design, construction, cost, and specification information provided in the catalog of technologies. Less experienced engineers, planners, and owners will benefit from the technology selection assistance portion of the system to assess the feasibility of technologies to address project requirements and constraints. The experience, technical ability, and judgment of the user will control both the extent and nature of utilization of the web-based information system (Hopple 1998). The intent of the system is to offer a means of evaluating a particular geoconstruction technology and to enable the user to determine where, when, and how a certain geoconstruction technology should be used (Terrel *et al.* 1979).

Geoconstruction Technologies

Geoconstruction technologies provide modification of site foundation soils or project earth structures to provide better performance under design and/or operational loading conditions (USACE 1999). The growth in geoconstruction technologies, products, systems, and engineering tools has been tremendous, with a very large body of knowledge and a large number of technologies available. Progress in this development has been chronicled by means of many conferences, workshops, papers and reports, too many to be cited herein.

However, a few comprehensive references that describe many of the technologies included in this web-based system are ASCE (1978, 1987, 1997), Chu *et al.* (2009), Elias *et al.* (2006), Holtz (1989), Mitchell (1981), Munfakh and Wyllie (2000), and Terashi and Juran (2000). The information system described herein builds upon these earlier works and provides a comprehensive reference for each geoconstruction technology. The web-based system allows this information to be easily accessible and publicly available.

A large number of geoconstruction technologies were initially identified at the start of system development. The number of technologies was winnowed to 46 based on their applicability to transportation-related projects. The technologies included in the system come from the following areas: ground improvement, geosynthetics, grouting, slope stabilization, soil reinforcement, soil stabilization including chemical and mechanical processes, and alternative/recycled materials. Excavation and replacement and traditional compaction are two traditional technologies included, as they are frequently utilized "base" technologies to which other technologies are often compared. The information system has intentionally avoided endorsing certain geoconstruction technologies over others and, to the extent possible, naming specific manufacturers and contractors.

The Web-Based Information System

The vital information available through the web-based information system allows for selecting, applying, designing, cost estimating, specifying, and monitoring construction of the 46 geoconstruction technologies. The web-based information system does not replace the judgment of the project engineer or user. The system does assist the user with selection and implementation of geoconstruction technologies for a specific project. The information system is a comprehensive toolkit of geotechnical information to address all phases of decision making to allow transportation projects to be built faster, to be less expensive, and/or to last longer.

Development of the information system began in Fall 2009. A constant cycle of review, commenting, and revision was interwoven into development with every revision resulting in a more usable, intuitive system developed by engineers for engineers. The Shewart cycle of Plan-Do-Check-Act describes the development of the system (Naik and Tripathy 2008). The "Plan" included establishment of the system objectives and outlining the process to deliver the results. The "Do" was the implementation of the plan. The "Check" assessed the system results and obtained decision maker input. The "Act" involved identification of changes and revisions required to improve the system. Eight reviews of the system with input from potential users during development provided valuable comments and suggestions. The reviews included state and federal transportation agency personnel, as well as academia, practitioners, and specialty contractors.

The overall concept of the information system is illustrated in Figure 7. The web-based system allows multiple users to access the technology information over the world-wide web. The information system is currently housed and maintained at Iowa State University. A screenshot of the homepage for the web-based system is provided in Figure 8.

The Operating System

After consideration of many different platforms and programming languages, the dynamic website was developed utilizing Adobe ColdFusion® server software in conjunction with a Microsoft Access® database. The combination of technologies allowed for the various pieces of the information system to be segregated into separate tables within a single database that could be dynamically queried via the web. The database is utilized to dynamically establish all the lists throughout the system, generate the details on each webpage unique to each technology, direct the system to the downloadable files, and contain the knowledge for the Interactive Selection Assistance Tool.

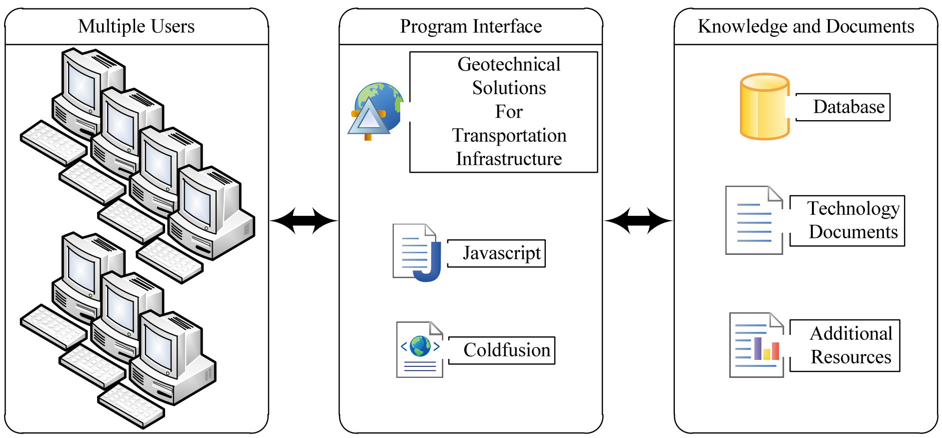


Figure 7. Information system overall concept

The programming was completed in the ColdFusion Markup Language (CFML). Additionally, the JavaScript programming language was incorporated to provide interactive site content and allow for live page updates based on user actions. The free, open-source JavaScript library, jQuery, was utilized to simplify the program's JavaScript coding in some instances, as well as extend its capabilities and ensure cross-browser compatibility as much as possible. An unforeseen challenge was developing a dynamic website compatible with the myriad of web browsers available to users. Dropdown boxes and other user features throughout the website were completed using a combination of Coldfusion and JavaScript.

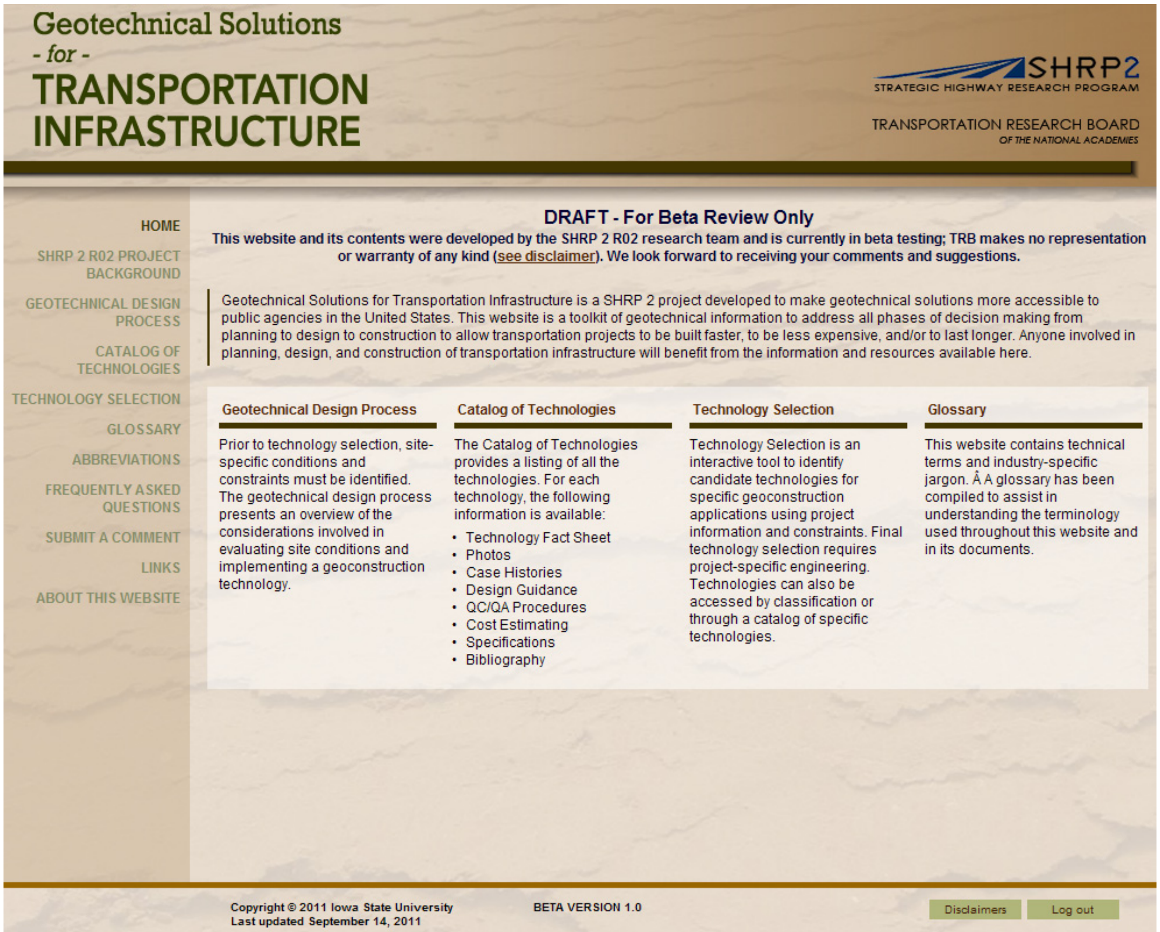


Figure 8. Screenshot of homepage for the web-based information system

Potential Users

A significant consideration throughout development was that the site should be beneficial to both technical and nontechnical users. The target audience for the system is primarily public agency geotechnical engineering personnel at local, state, and federal levels. However, civil/structural, construction, pavement, and construction engineers in consulting, contracting, and academia will also find the system useful, as will transportation managers and decision makers. Although focused on the transportation industry, the technologies in the system can be applied equally well to non-transportation projects, and thus the system should have broad appeal to the geotechnical community. Although the description of the user seems intuitive, the process of system development revealed that the broad range of potential users resulted in unique considerations in all phases and areas of system development. For example, technical terms were intentionally avoided in the *Technology Fact Sheets* and the first few steps of the Interactive Selection Assistance Tool in order to allow nontechnical users to learn about geoconstruction technologies for different types of transportation applications.

All users should acknowledge that a geotechnical information system deals with subject matter of realistic complexity and requires a considerable amount of human experience (Jackson 1999). The system layout was designed and developed such that experienced users can access all the required information, but that inexperienced users will recognize when additional support should be sought.

Main Components of the Web-Based Information System

The four main components of the information system are Geotechnical Design Process, Glossary, Catalog of Technologies, and Technology Selection. The inter-relationship of the four primary components with the other features of the system is illustrated in Figure 9. The dissemination of information through the Catalog of Technologies provides the mechanism to facilitate technology transfer to everyday practice. One of the goals of the Technology Selection component is to refer the user to the appropriate Individual Technology Information webpage within the Catalog of Technologies. The other features of the website, such as the Project Background, Frequently Asked Questions, Submission of Comments, Links, and About This Website, support the four primary components and usability of the website. The details of this development are summarized in the web-based system development report (Douglas *et al.* 2012). After a brief description of the Geotechnical Design Process and Glossary, extended discussions of the Catalog of Technologies and Technology Selection are provided.

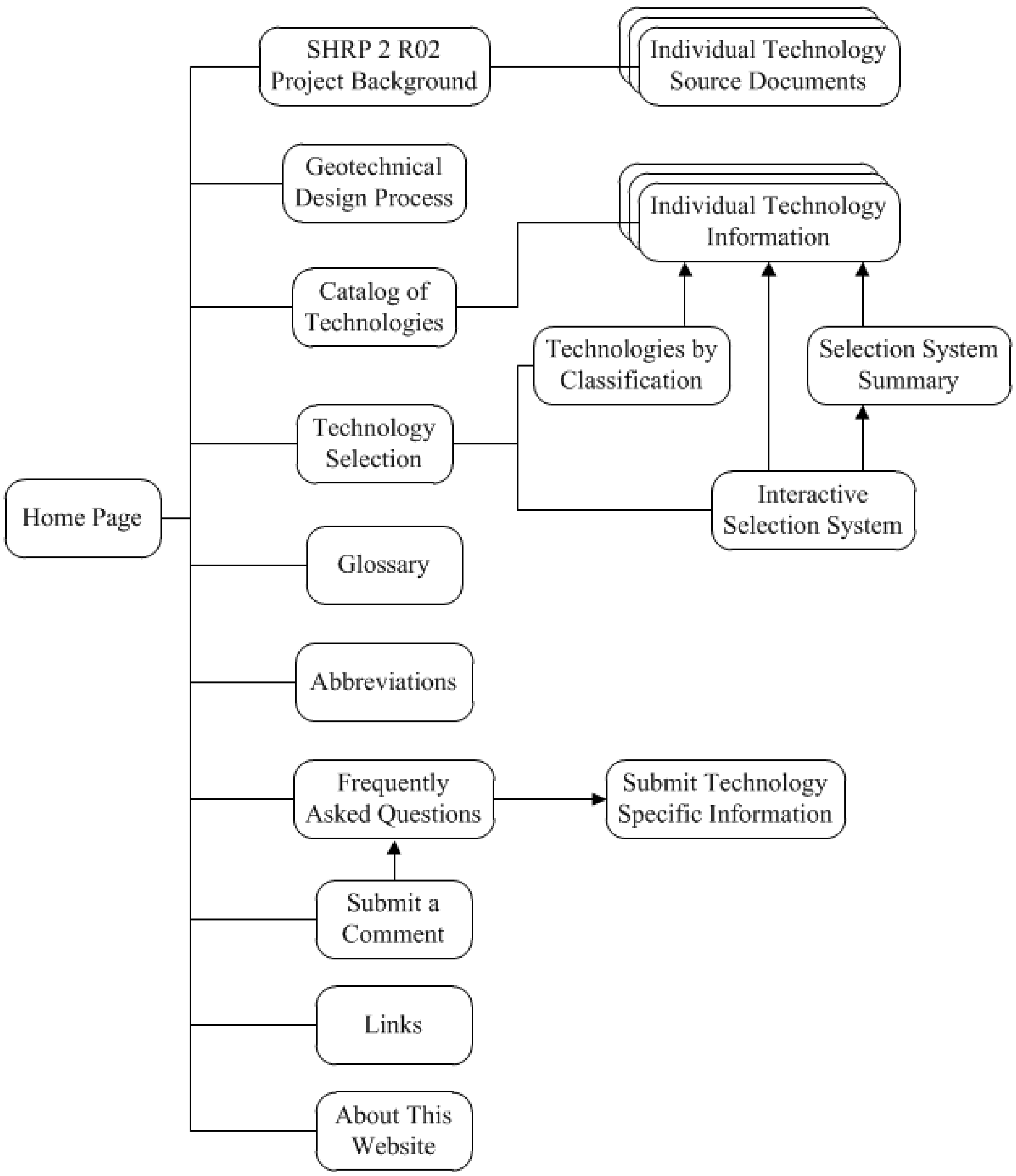


Figure 9. Overall map of the web-based information system

Geotechnical Design Process

Prior to considering a geoconstruction technology for utilization on a project, site-specific conditions and constraints must be identified in relation to the project requirements. The Geotechnical Design Process presents an overview of the considerations involved in evaluating site conditions and implementing a geoconstruction technology. The project engineer is responsible for determining the appropriate project-specific geotechnical design process. The technology-specific information provided in the Catalog of Technologies does identify key site conditions and design parameters to be determined as part of the Geotechnical Design Process. The system guides the user to Federal Highway Administration (FHWA) documents on the review of geotechnical reports, evaluation of soil and rock properties, subsurface investigation, and instrumentation and monitoring. Additionally, links to several state transportation agency (STA) geotechnical design manuals are provided.

Glossary

During the development of the system, it was realized that a large number of technical terms were used and that in some cases different technologies used terms in different ways. Thus, a Glossary was compiled and a webpage included to assist the user in understanding the terminology used throughout the website and in its documents. The Glossary webpage is provided in an alphabetical listing sequence with a linked system of the alphabet for ease of use. The definitions of the terms refer to existing documents where possible. Where a clear definition did not exist, the terms were defined as utilized in this system. Several of the terms have similar but slightly different published definitions from various sources. Links are also provided to publicly available compilations of definitions available through the world-wide web, as well as links to purchase copyrighted definitions. An Abbreviations webpage was developed to supplement the Glossary and assist the user with deciphering the myriad of abbreviations utilized in the practice of applying geoconstruction technologies.

Catalog of Technologies

The Catalog of Technologies webpage provides a listing of the 46 geoconstruction technologies in the system. The name of each technology is a linked button that takes the user to a Technology Information webpage for that technology, as illustrated in Figure 10. The Technology Information webpage represents the technology transfer for each geoconstruction technology included in the system. Included on each Technology Information webpage is a series of ratings. Technology ratings were developed through the completion of a qualitative assessment by the project team to rate the technologies according to Degree of Technology Establishment in the U.S., Potential Contribution to Rapid Renewal of Transportation Facilities, Potential Contribution to Minimal Disruption of Traffic, and Potential Contribution to Production of Long-Lived Facilities.

From the individual Technology Information webpage, the user can access the following documents, which are generally provided as portable document format (PDF) files: *Technology Fact Sheets*, *Photographs*, *Case Histories*, *Design Guidance*, *Quality Control/Quality Assurance* (QC/QA), *Cost Information*, *Specifications*, and *Bibliography*.

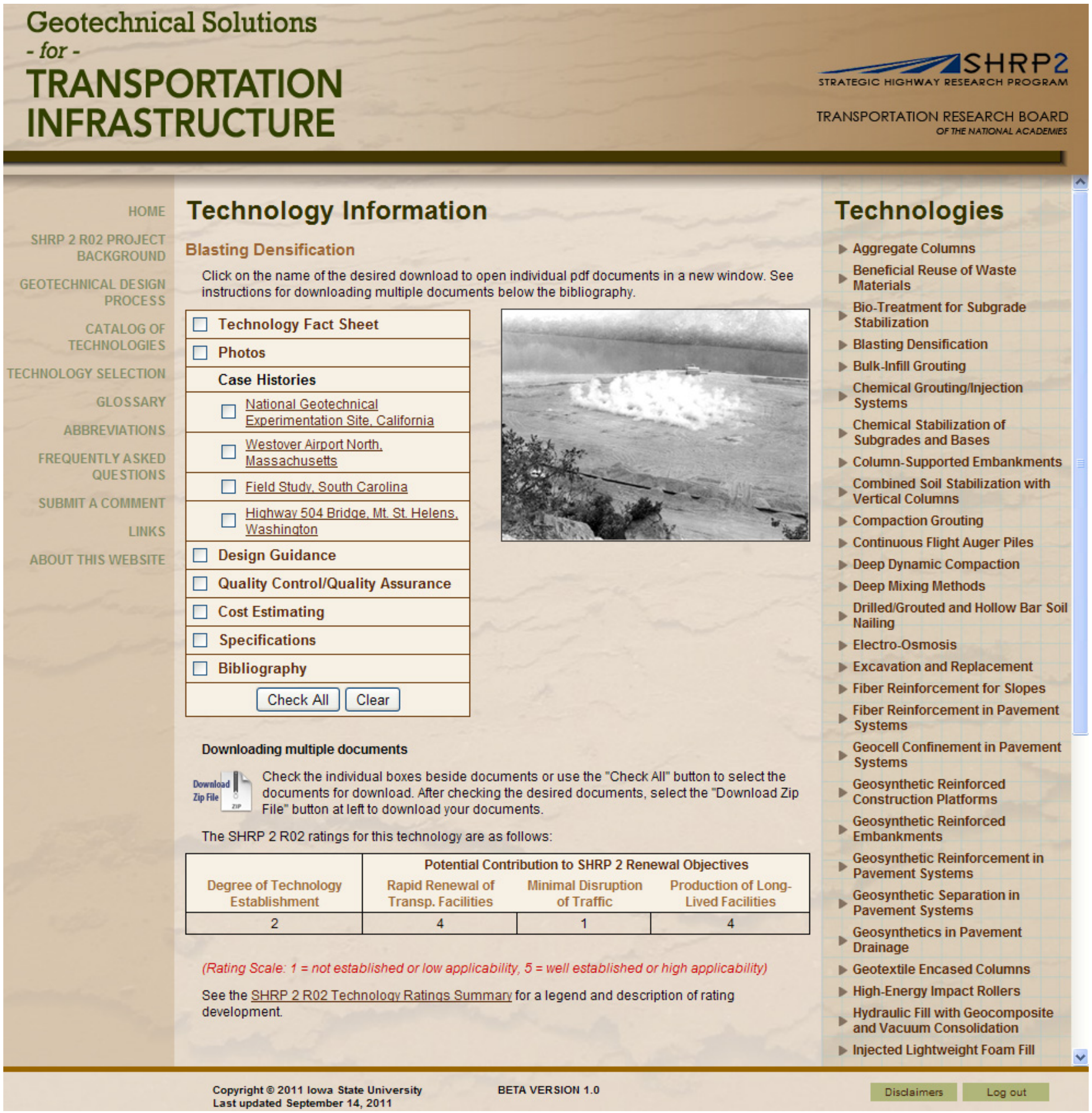


Figure 10. Screenshot of individual technology webpage with documents available for download

The documents available for each technology are the result of completing Comprehensive Technology Summaries (CTS), Design Procedure Assessments, QC/QA Assessments, and Cost Evaluations for each of the technologies. For reference, these summaries, assessments, and evaluations can be found through the Project Background webpage. However, these work products were intentionally separated because the documents provided on the Technology Information webpages provide concise summaries of those assessments and evaluations completed during system development. CTS development entailed data mining to produce an in-depth technology overview that included advantages, potential disadvantages, applicable soil types, depth/height limits, groundwater conditions, material properties, project-specific constraints, equipment needs, and environmental considerations. Assessments were completed for design, QC/QA, and specifications to identify key material for each technology. The downloadable documents available on the Technology Information webpages resulted from the completion of these assessments and evaluations for each technology.

The *Technology Fact Sheet* is a two-page summary information sheet that provides basic information on the technology, including basic function, general description, geologic applicability, construction methods, transportation applications, complementary technologies, alternate technologies, potential disadvantages, example successful applications, and key references. The *Photos* show pictorially the equipment or methods used in the technology and can be valuable to gain a perspective on the technology. The *Case Histories* provide a summary of project(s) that were preferably conducted in the U.S. by a STA and contain project location, owner, a project summary, performance, and contact information. The first page (of two) of an example case history is shown in Figure 11 and is typical of the downloadable documents for each technology. For some technologies new to the U.S., the case histories were developed from projects in other countries.

Mechanically Stabilized Earth Walls
SeaTac Airport Runway Extension Project

Location: Minneapolis, MN

Owner:
Minnesota Department of Transportation

Contractor:
Ames Construction, Inc.

Engineers:
MnDOT and
SRF Consulting Group, Inc.

Years Constructed:
2008 and 2009

Photo courtesy of R. Berg

Project Summary/Scope:
MSE walls were used extensively on the Crosstown Project located in Minneapolis, Minnesota. The walls were used to widen existing roadways, construct new ramps, and construct new bridge approaches. The project enlarged and streamlined the I-35W and Minnesota Highway 62 interchange. This is a heavily traveled roadway in a congested urban area. Several bridges were widened and several new bridges were constructed as part of this project. The detailed wall designs and the wall components were supplied by a Minnesota Department of Transportation (MnDOT) pre-approved MSE wall vendor. Design followed the ASD method (AASHTO 17th Edition 2002). Most of the walls were constructed during the 2008 and 2009 construction seasons.

Approximately 300,000 ft^2 (28,000 m^2) of MSE walls were constructed in 24 separate walls. Typical wall heights were approximately 25 to 30 feet (7.5 to 9 meters), and the maximum wall height was 45 feet (14 m). The MSE walls were faced with architectural segmental precast panels and reinforced with steel bar mats. Facing panels were painted after wall construction. The architectural relief included false columns on the long and tall walls, as shown in above photograph.

The reinforced wall fill was an angular, well-graded sand. The walls were designed for a 100-year life. Many walls had traffic barriers on top of the reinforced zones. The barriers were designed by the MnDOT project design consultant. A geomembrane was specified and installed across the top of the reinforced zones to prevent or minimize infiltration of de-icing salt runoff into the reinforced fill.

R02 GEOTECHNICAL SOLUTIONS FOR SOIL IMPROVEMENT,
RAPID EMBANKMENT CONSTRUCTION, AND
STABILIZATION OF PAVEMENT WORKING PLATFORM

SHRP2
STRATEGIC HIGHWAY RESEARCH PROGRAM

PROJECT CASE HISTORY

Figure 11. **Example *Case History***

The *Design Guidance* summarizes the recommended design procedures for the technology. In cases where a well-established procedure (e.g., a FHWA manual) exists, that procedure is recommended. In cases of technologies with multiple proposed procedures but with no established preferred procedure, the assessment led to a recommendation of which procedure(s) to use. The *Design Guidance* identifies typical considerations during design in the following areas: performance criteria/indicators, subsurface conditions, loading conditions, material characteristics, and construction techniques. The *QC/QA Procedures* document provides a summary of recommended procedures for each technology. The recommended QC/QA procedures resulted from an assessment of the current state of the practice of each technology. For a few technologies, design and/or QC/QA procedures were refined and improved within the project work.

For most technologies, two documents are available to assist with estimating costs. The first, a downloadable document from the Technology Information webpage titled *Cost Information*, provides an explanation of the cost items specific to the technology, generally emanating from the pay methods contained in identified specifications. Project-specific conditions and their impact on cost are discussed in the explanation. The *Cost Information* compiles available regional and cost numbers from STA bid tabs or national databases when available. For technologies with scarce or no STA cost history, the *Cost Information* provides a discussion of important considerations for the technology when estimating costs. The second document consists of an Excel spreadsheet developed to preliminarily estimate costs for the use of the technology. The spreadsheet can only be accessed through a link in the *Cost Information* document in order to force the user to access the cost discussion prior to developing a preliminary estimate. An example spreadsheet is shown in Figure 12. The Excel spreadsheet could not be prepared for some technologies due to insufficient information availability. The spreadsheet can be modified by the user to estimate specific project cost based on either a preliminary or final design. Many decisions in transportation are cost driven. In order to avoid quick elimination of technologies on cost, simplified “rule of thumb” costs were avoided in the *Cost Information* documents. The cost spreadsheets require that a preliminary assessment or design be completed prior to estimating costs. A valid comparison of technology costs can only be completed after a preliminary design has been developed. The information system provides the user with the tools to complete a preliminary design and subsequent cost analysis that captures the technology-specific costs of implementation and construction.

A *Specifications* document is provided for each technology. The *Specifications* vary from identification of an existing specification that can be utilized for future projects, to a specification developed during system development, to a description of topics for consideration when developing a specification for a specific technology. The final document available for each technology is a *Bibliography* compiled during the research project. In order to assist sorting the references in the *Bibliography* document, a reference matrix with 22 categories is provided to highlight the information in the reference, such as technology overview, design procedure, construction methods, cost, specification, QC/QA, and case history to name a few.

Conceptual Estimating Tool - Prefabricated Vertical Drains and Fill Preloading

Notes to User:

A. This estimating tool is provided as a means to perform an initial project scoping estimate. Use for any other purpose is strongly discouraged. The accuracy and reliability of the estimated costs are highly dependent upon the user inputs, care should be taken to adjust inputs for specific project characteristics. The user assumes all risks associated with the cost estimates produced by this estimating tool.

B. Guidance on unit cost ranges and potential impacts on cost is provided in the cost information summary for each technology. Users are responsible for determining appropriate unit costs.

C. Cells highlighted in "burnt orange" require user input.

D. Cells with "maroon" colored text are automatically calculated, but may be manually overridden by the user.

1. Calculate the Surface Area Where PVDs are to be Installed

Length (ft):	2,000
Width (ft):	150
Area (ft^2):	300,000

2. Estimate the Total Quantity of PVDs to be Installed

Design output information required - Preliminary PVD grid spacing and average depth of installation are necessary for this step

Estimated Longitudinal Grid Spacing of PVDs (ft):	5.00
Estimated Transverse Grid Spacing of PVDs (ft):	5.00
Number of PVDs to be Installed:	12,431
Average Depth of PVD Installation (ft):	50
Total Quantity of PVDs (lf):	621,550

3. Estimate the Drainage Layer Quantity - Complete One of the Options Listed

Design output information required - Drainage layer type and volume/spacing characteristics are necessary for this step

Granular Material Option:	
Thickness of Granular Drainage Layer (in):	-
Approximate In-Place Density of Granular Drainage Layer (lb/ft^3):	110
Total Quantity of Granular Drainage Layer (ton):	-
OR	
Horizontal Strip Drain Option:	
Estimated Longitudinal Spacing of Horizontal Strip Drain (ft):	5
Total Quantity of Horizontal Strip Drain (lf):	60,000

4. Optional Depending Upon Soil Conditions - Estimate the Quantity of Augering Through Stiff Upper Soil Strata

Estimated Thickness of Stiff Soil Requiring Augering (ft):	10
Number of Augered Holes (ea):	12,431
Total Quantity of Augered Holes (lf):	124,310

5. If Needed, Estimate the Materials Required for an Initial Working Platform

Length (ft):	2,000
Width (ft):	150
Quantity of Geosynthetic for a Working Platform (yd^2):	33,333
Optional, Thickness of Granular Layer for Working Platform (in):	6
Optional, Estimated Density of Granular Material for Working Platform (lb/ft^3):	120
Total Quantity of Granular Material for Working Platform (ton):	9,000

6. Optional, Estimate the Surcharge Volume Required

Design output information required - Surcharge volume is dependent upon the desired settlement rate

Average Surcharge Length (ft):	2,000
Average Surcharge Width (ft):	150
Average Surcharge Height (ft):	10
Total Surcharge Volume (yd^3):	111,111

7. Estimate Additional Embankment due to Settlement and/or the Surcharge Volume to be Removed (wasted)

Design output information required - Estimated amount of settlement is needed

Surcharge removal is reduced by the estimated settlement - there is no additional cost attributable to PVDs when the surcharge removal can be utilized in an embankment on the project

Estimated Settlement (ft):	4
Optional, Total Surcharge Volume to be Wasted (yd^3):	66,667

8. Estimated Cost of PVDs - Refer to Cost Information Summary for Typical Unit Cost Ranges and Impacts on Unit Prices

	Unit Cost	Quantity	Cost
Optional, Geosynthetic for Working Platform (yd^2):	\$ 2.75	33,333	\$ 91,667
Optional, Granular Material for Working Platform (ton):	\$ 10.00	9,000	\$ 90,000
PVD Unit Price (\$/lf):	\$ 1.00	621,550	\$ 621,550
Mobilization (lump sum):	\$ 15,000.00	1	\$ 15,000
Granular Drainage Layer (\$/ton):	\$ 15.00	-	\$ -
OR			
Horizontal Strip Drain (\$/lf):	\$ 1.00	60,000	\$ 60,000
Optional Pre-Augered Holes (\$/lf):	\$ 8.00	124,310	\$ 994,480
Surcharge Embankment (\$/yd^3):	\$ 4.00	111,111	\$ 444,444
Additional Embankment Due to Settlement, Applies if No Surcharge is Constructed or if Settlement Exceeds Surcharge Height (\$/yd^3):	\$ 4.00	-	\$ -
Surcharge Excavation (\$/yd^3) and (yd^3):	\$ 4.00	66,667	\$ 266,667
Estimated Total Cost of PVD Installation:			**\$ 2,583,808**
Estimated Unit Cost of PVD Installation for Area Treated (\$/ft^2):			**\$ 8.61**

Figure 12. Example conceptual estimating tool

Technology Selection

Technology Selection was developed to aid in identifying potential geoconstruction technologies for a user-defined set of project conditions. Technology Selection contains both a listing of the technologies sorted by classification and a dynamic, Interactive Selection Assistance Tool. After the user identifies potential technologies, the Technology Information webpage can be accessed, which includes information necessary for additional screening (i.e., depth limits, applicability to different soil types, acceptable groundwater conditions, applicability to different project types, ability to deal with project-specific constraints, general advantages/disadvantages, etc). The aim of Technology Selection is to point the user back to the technology specific information found in the Catalog of Technologies.

An experienced engineer can access solutions according to particular classifications or categories of problems. Various categories of ground improvement technologies have been presented by many authors, as previously cited in the references in the Geoconstruction Technologies section. For technologies included in the information system, the technologies are grouped by the following classifications: Earthwork Construction, Soft Ground Drainage & Consolidation, Densification of Cohesionless Soils, Construction of Vertical Support Elements, Embankments Over Soft Soils, Lateral Earth Support, Cutoff Walls, Liquefaction Mitigation, Increased Pavement Performance, Void Filling, and Sustainability.

The Interactive Selection Assistance Tool provides the user the opportunity to assess technologies based on several applications. The uniqueness of the Interactive Selection Assistance Tool is the approach of assigning a geoconstruction technology on the basis of application. The first decision in the tool, as illustrated in Figure 13, is to select one of the four potential applications: Construction over Unstable Soils; Construction over Stable or

Stabilized Soils; Geotechnical Pavement Components including Base, Subbase, and Subgrade; and Working Platforms. Each application results in a unique set of queries to winnow the possible technologies for each application. Pop-up help windows appear next to each query to explain the purpose or intent of the posed query and to assist the user in determining the proper selection.

The Interactive Selection Assistance Tool is a knowledge-based system. Special programming formed the logic, and the knowledge is contained in a series of tables within the database. Each selection queries a database column and utilizes a nested if…then statement to sort the appropriate technologies. A significant benefit of the rule-based approach is the sharing of knowledge, especially when the knowledge is not the type of knowledge typically published in scholarly publications (Spring *et al.* 1991). The knowledge for identifying potentially applicable technologies to a set of geotechnical and project conditions was initially developed from the summaries and assessments for each of the technologies. The knowledge was then evaluated by experts on the research team and advisory board during final development. The process of the elimination of technologies is best described as a heuristic process. Intuition, experience, and judgment can be utilized to develop heuristic rules (Ignizio 1991). Heuristics such as "Do you know a related problem" requires the recollection of previous projects (Cheng *et al.* 2008). These recollections from experts were utilized to refine the knowledge base for the selection system.

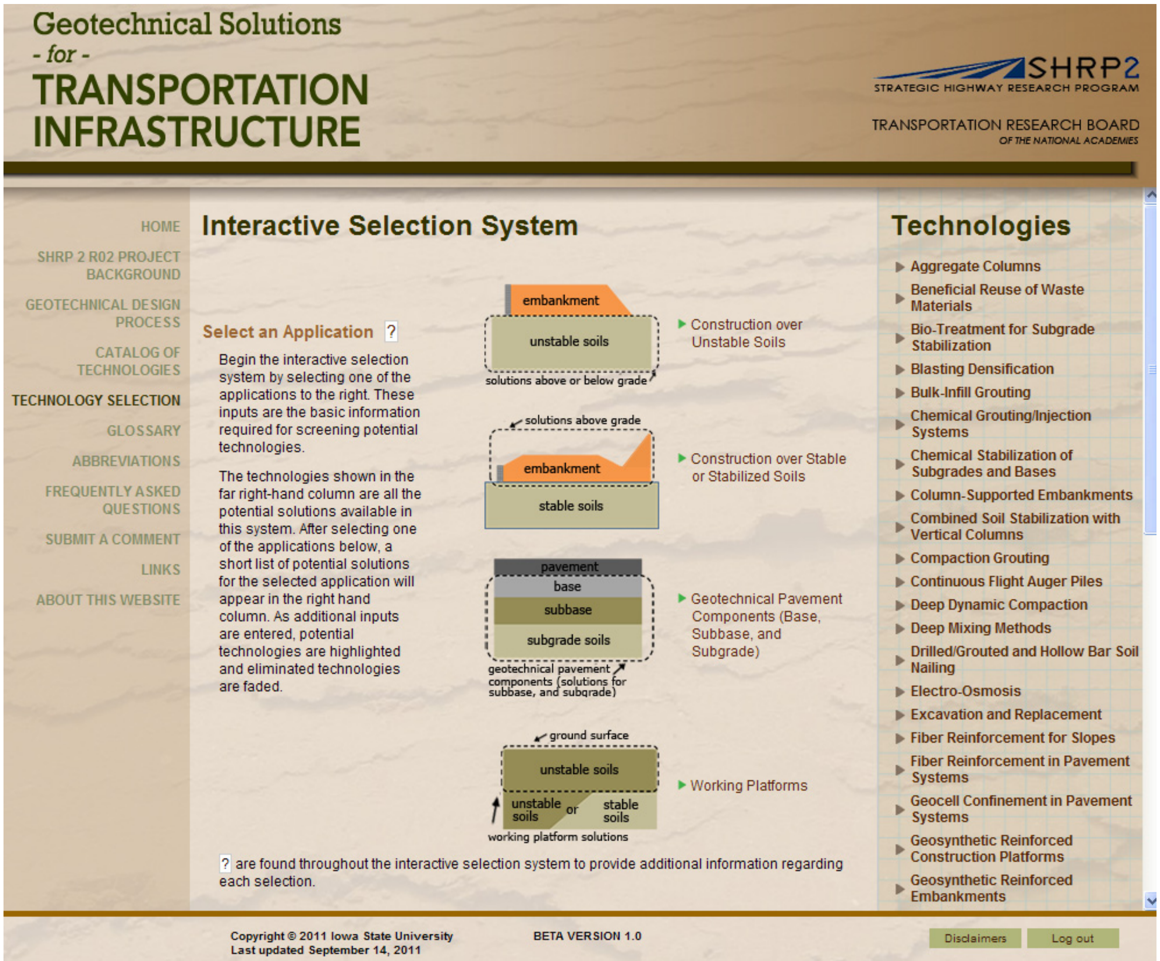


Figure 13. Interactive Selection Tool webpage

After completion of the Interactive Selection Assistance Tool, the user has the opportunity to document the results through the creation of a PDF file, as illustrated in Figure 14. The output shows the responses to the queries and the potentially applicable technologies. The output also allows the users to include their name and project information. The date is automatically generated on the output. For further information, the user can access the individual technology catalog pages from the PDF.

Like most geotechnical analytical solutions, the results of the analysis must be measured against the opinion of an experienced geotechnical engineer practicing in the local area of the project. The Interactive Selection Assistance Tool does not replace the project Geotechnical Engineer. The Geotechnical Engineer's "engineering judgment" should be the final selection process, which takes into consideration the following: construction cost, maintenance cost, design and quality control issues, performance and safety (pavement smoothness, hazards caused by maintenance operations, potential failures), inconvenience (a tangible factor, especially for heavily traveled roadways or long detours), environmental aspects, and aesthetic aspects (appearance of completed work with respect to its surroundings) (Johnson 1975; Holtz 1989).

Limitations of the Web-Based Information System

The abundance of knowledge available through the web-based information system can easily be misused by inexperienced personnel. Marr (2006) developed five "take home messages" concerning geotechnical engineering and judgment in the information age that adequately address how a user should approach the information system. (1) Engineering judgment without relevant experience is weak. (2) Engineering judgment without relevant data is foolish. (3) Good judgment needs good data and evaluated experience. (4) Good judgment is essential for the effective use of information technology tools. (5) Good judgment is central to geotechnical engineering, even in the information age.

The current, initial breadth of this information system is limited to 46 geoconstruction technologies, and all documents are current as of the time of system development. The technologies included were primarily defined by the scope of the development project.

Geotechnical Solutions for Transportation Infrastructure (BETA Version 1.0)

Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform

Technology Selection Results

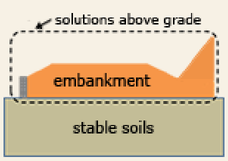
This page provides documentation of the results of the interactive selection system and provides a means of allowing the user access to the technology catalog page without losing the results of the technology selection system. The listed potential technologies are linked to the technology catalog page for each technology which will open in a new window.

The application of this system is the responsibility of the user. It is imperative that the responsible engineer/user understand the potential accuracy limitations of the program results, independently cross checks those results with other methods, and examines the reasonableness of the results with engineering knowledge and experience.

Please refer to the document User's Guide to the Information and Guidance System for the constraints, intended uses, and limitations of the Technology Selection portion of this website.

Project Characteristics

The project and site information input into the selection system is summarized below.



Selected Application: *Construction over Stable or Stabilized Soils*

Purpose of Application: *Slope Stabilization and Earth Retention*

Slope Characteristic: *Existing, Cut, or Cut/Fill Slope*

Potential Technologies

The potential technologies as a result of the project and site information are shown below.

Technology	Degree of Establishment*	Potential Contribution to SHRP 2 Renewal Objectives: Rapid Renewal*	Minimal Disruption*	Long-Lived Facilities*
Drilled/Grouted and Hollow Bar Soil Nailing	4	4	3	4
Mechanically Stabilized Earth Wall System	5	3	2	4
Screw-in Soil Nailing	2	4	3	3
Shoot-in Soil Nailing	3	4	4	3
Shored Mechanically Stabilized Earth Wall System	3	3	3	4

*See the SHRP 2 R02 Technology Ratings Summary for a legend and description of rating development.

Figure 14. Output from Interactive Selection Tool

Conclusions

A knowledge base has been compiled for 46 geoconstruction technologies, and a web-based information system has been developed to facilitate and organize this knowledge so that informed decisions can be made. The system assists in the selection and implementation of a suitable geoconstruction technology. Detailed information provides for optimization of design, cost estimating, specifying, constructing, and assuring quality to meet specific project requirements. Even with the wealth of information provided in the system, proper application of a geoconstruction technology requires extensive background knowledge of available ground treatment technologies and careful evaluation of several factors. These factors include understanding the functions of the method, utilization of several selection criteria, the use of appropriate design procedures, implementation of the right technologies for QC/QA, and consideration of all relevant cost components and environmental factors. The technical information provided in the information system combined with the engineering judgment of the user will result in transportation projects that are built faster, cost less, and last longer.

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CHAPTER 4. SELECTION ASSISTANCE FOR THE EVALUATION OF GEOCONSTRUCTION TECHNOLOGIES

Modified from a paper submitted to the *Journal of Geotechnical and Geological Engineering*, published by Springer

S. Caleb Douglas[8 9], P.E., and Vernon R. Schaefer[10 11], P.E., and Ryan R. Berg[11 12], P.E., D.GE

Abstract

A new comprehensive, web-based information system summarizes 46 geoconstruction technologies, or ground improvement methods, applicable to transportation infrastructure from the following areas: geosynthetics, geotextiles, ground improvement, grouting, slope stabilization, soil reinforcement, soil stabilization, and alternative/recycled materials. Selection assistance was developed as part of the overall information system to aid the user in identifying potential geoconstruction technologies for a project-specific set of conditions. A knowledge base to assist a user in evaluating the current status of each

[8] Ph.D. candidate, Civil Engineering, Iowa State University, Ames, IA 50010-3232. E-mail: calebd@iastate.edu
[9] Primary researcher and author, author for correspondence
[10] Professor of Civil Engineering, Iowa State University, Ames, IA 50010-3232. E-mail: vern@iastate.edu
[11] Secondary author
[12] Ryan R. Berg & Associates, Inc., Woodbury, MN, 55125-3504. E-mail: ryanberg@att.net

technology with regard to the U.S. practice and the potential applications for each technology is described in detail. Selection assistance includes qualitative ratings for each technology, a listing of the technologies sorted by classification or desired improvement, and a dynamic Interactive Selection Tool. After assisting the user in identifying a short list of potential technologies, the user can access the technology-specific data in the information system to further evaluate the technologies. Engineers, planners, and owners will be able to utilize the entire information system to assess the feasibility of technologies to address project-specific requirements and constraints.

Introduction

A web-based information system was developed for the forty-six ground improvement methods, geotechnical materials, systems, and technologies (referred to as "geoconstruction technologies") listed in Table 20. The primary value of the web-based information system is that it collects, synthesizes, integrates, and organizes a vast amount of critically important information about geotechnical solutions in a system that makes the information readily accessible to the user. The web-based information system will be a valuable tool for engineers, planners, and transportation officials to utilize when evaluating potential geoconstruction technologies. No system like this currently exists, either in hard form or through a programmed system. Comprehensive, automated systems for geoconstruction technologies have been developed in the past, but the systems are either currently unsupported or publicly unavailable (Chameau and Santamarina 1989; Motamed *et al.* 1991; Sadek and Khoury 2000).

Table 20. Geoconstruction technology list and ratings

Technology	Technology ratings			
	Degree of technology establishment	Rapid renewal	Minimal disruption of traffic	Long-lived facilities
Aggregate columns	4	3	1	4
Beneficial reuse of waste materials	3	2	1	3
Bio-treatment for subgrade stabilization	1	3	3	3
Blasting densification	3	3	2	4
Bulk-infill grouting	3	4	4	4
Chemical grouting/ injection systems	3	3	4	4
Chemical stabilization of subgrades and bases	5	4	2	4
Column-supported embankments	3	5	1	4
Combined soil stabilization with vertical columns	2	3	1	4
Compaction grouting	4	3	3	3
Continuous flight auger piles	4	4	1	4
Deep dynamic compaction	5	4	1	4
Deep mixing methods	3	4	1	4
Drilled/grouted and hollow bar soil nailing	4	4	3	4
Electro-osmosis	2	2	5	4
Excavation and replacement	5	2	1	4
Fiber reinforcement in pavement systems	2	3	2	4
Geocell confinement in pavement systems	1	3	2	4
Geosynthetic reinforced construction platforms	5	4	2	3
Geosynthetic reinforced embankments	5	4	2	4
Geosynthetic reinforcement in pavement systems	4	4	2	4
Geosynthetic separation in pavement systems	4	4	2	4
Geosynthetics in pavement drainage	4	4	3	4

Table 20. (continued)

Technology	Technology Ratings: Degree of technology establishment	Rapid renewal	Minimal disruption of traffic	Long-lived facilities
Geotextile encased columns	1	3	1	3
High-energy impact rollers	2	4	2	4
Hydraulic fill with geocomposite and vacuum consolidation	1	2	1	3
Injected lightweight foam fill	2	3	3	3
Intelligent compaction	2	3	2	4
Jet grouting	4	4	2	4
Lightweight fills	5	5	3	3
Mechanical stabilization of subgrades and bases	5	2	1	4
Mechanically stabilized earth wall systems	5	3	2	4
Micro-piles	4	3	2	3
Onsite use of recycled pavement materials	4	3	2	3
Partial encapsulation	3	3	2	4
Prefabricated vertical drains (PVDs) and fill preloading	5	3	1	4
Rapid impact compaction	2	4	1	3
Reinforced soil slopes	5	3	2	4
Sand compaction piles	2	4	1	3
Screw-in soil nailing	2	4	3	3
Shoot-in soil nailing	3	4	4	3
Shored mechanically stabilized earth wall system	3	3	3	4
Traditional compaction	5	2	1	3
Vacuum preloading with and without PVDs	2	3	1	4
Vibrocompaction	5	4	1	4
Vibro-concrete columns	3	4	1	4

The web-based information system is best described as a decision support system. The web-based system will be utilized by both technical and nontechnical personnel for all types of transportation-related projects. Each project will have a unique set of field, loading, and boundary conditions. A decision support system shifts the role of computers from one of generating data and information to a more advanced function of supporting, in a variety of ways, decision making in complex and ill-structured task settings (Hopple 1998). After consideration of several different platforms and programming languages, the dynamic website was developed utilizing Adobe ColdFusion® software in conjunction with a Microsoft Access® database. The combination of technologies allowed for the various pieces of the information system to be segregated into various tables within a single database that could be dynamically queried via the web. A screenshot of the homepage for the web-based system is provided in Figure 15.

The four main components of the information system are the Geotechnical Design Process, Glossary, Catalog of Technologies, and Technology Selection. The dissemination of information through the Catalog of Technologies provides the mechanism for detailed technology transfer to everyday practice. From the Technology Information webpage, as shown in Figure 16, the following documents can be accessed for each technology: *Technology Fact Sheet*, *Photographs*, *Case Histories*, *Design Guidance*, *Quality Control/Quality Assurance*, *Cost Information*, *Specifications*, and *Bibliography*. The details of the development of the information system are summarized in the web-based system development report (Douglas *et al.* 2012).

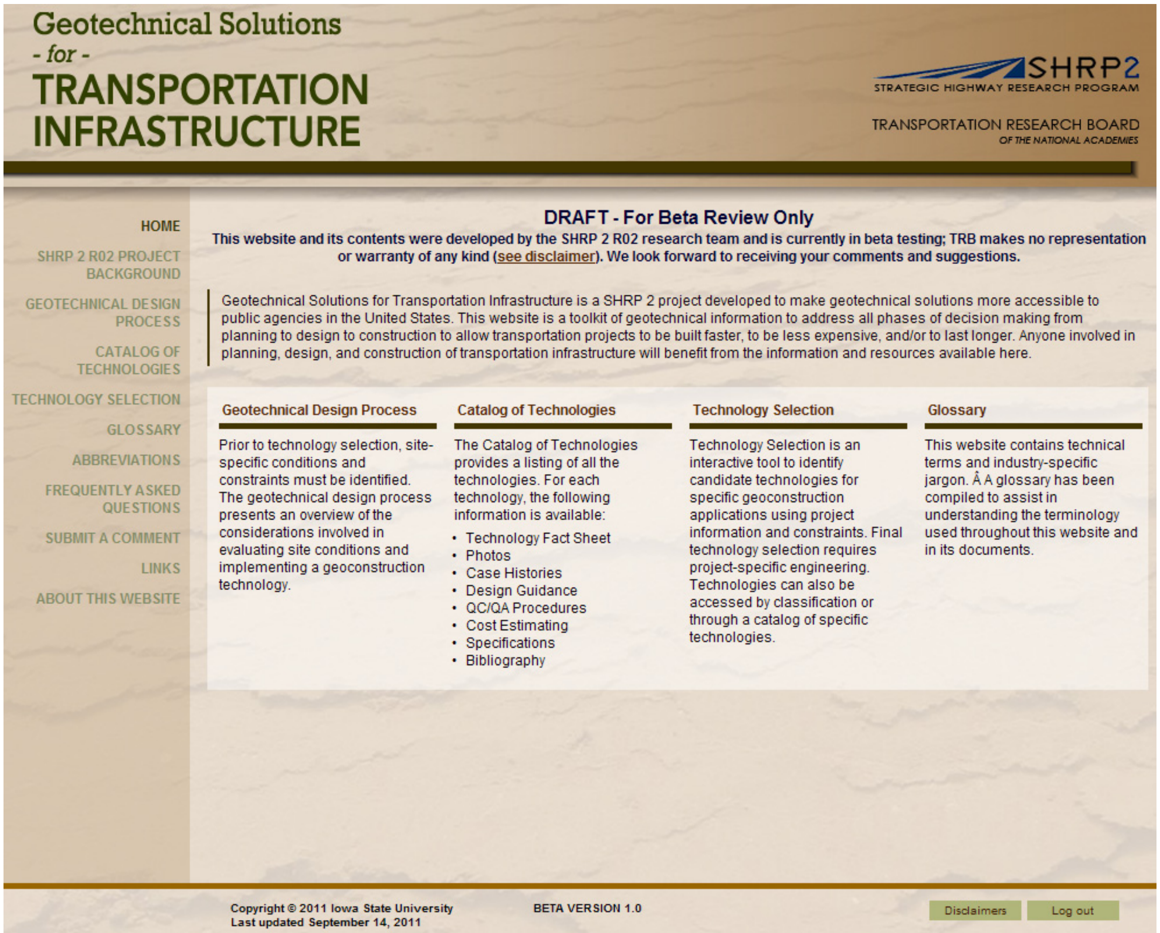

Figure 15. Screenshot of homepage for the web-based information system

The focus of this paper is the selection assistance portion of the information system, which aids the user in identifying potential geoconstruction technologies for a project-specific set of conditions. The following three aspects support a user in the selection of an appropriate geoconstruction technology:

- Provide qualitative ratings for each technology.
- Lead the user to a short list of unranked applicable technologies.

- Point the user to the appropriate technology-specific webpage within the Catalog of Technologies to facilitate easy and quick access to detailed information necessary for additional screening and a project-specific determination.

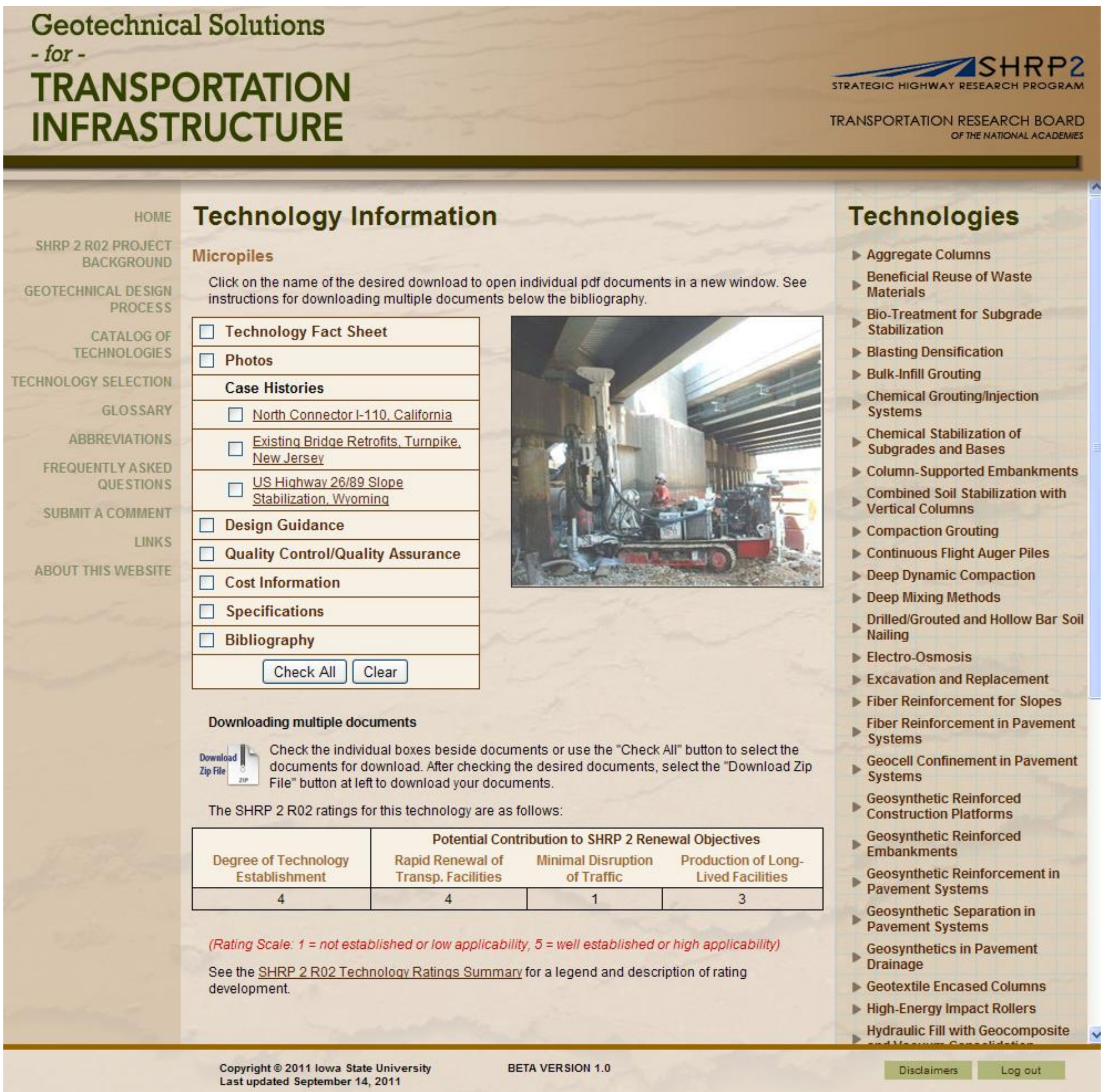

Figure 16. Screenshot of individual technology webpage

As illustrated in Figure 16, the qualitative ratings are integrated throughout the information system. The Technology Selection component of the web-based system includes two parts, a listing of the technologies sorted by classification and a dynamic Interactive Selection Tool. The Interactive Selection Tool was developed to guide the user through the process of identifying potential geoconstruction technologies for a unique set of project conditions. The details presented in this paper are automated in the web-based information system. An overview with many of the specifics is provided for the Interactive Selection Tool, but every detail of the tool is not presented.

Geoconstruction Technologies

Geoconstruction technologies provide modification of site foundation soils or project earth structures to provide better performance under design and/or operational loading conditions (USACE 1999). A large number of geoconstruction technologies was initially identified at the start of system development and was winnowed to 46 based on the technologies' applicability to transportation-related projects in the U.S. The information system has intentionally avoided, where possible, endorsing certain geoconstruction technologies over others and naming specific manufacturers/contractors.

The growth in geoconstruction technologies, products, systems, and engineering tools has been tremendous, with a very large body of knowledge and large number of technologies available. A few comprehensive references that describe many of the technologies and their applicability to project-specific conditions are ASCE (1978, 1987, 1997), Mitchell (1981), Holtz (1989), Munfakh and Wyllie (2000), Terashi and Juran (2000), Charles and Watts (2002), Elias *et al.* (2006), and Chu *et al.* (2009).

Considerations for Development of Selection Assistance

The process of selection of a particular geoconstruction technology initially appears to be a straightforward undertaking. However, the development of selection assistance revealed the complexity of the decision of selecting a geoconstruction technology. Some areas that required consideration were potential users, the elimination of technologies, and the development of the knowledge base.

Potential Users

A significant consideration throughout development was that the website should be beneficial to both technical and nontechnical users. The target audience for the system is primarily public agency geotechnical engineering personnel at local, state, and federal levels. However, civil/structural, construction, pavement, and construction engineers in consulting, contracting, and academia will also find the system useful, as will transportation managers and decision makers. Although focused on the transportation industry, the technologies in the system can be applied equally well to non-transportation projects, and thus the system should have broad appeal to the geotechnical community. Although the description of the user seems intuitive, the process of system development revealed that the broad range of potential users resulted in unique considerations in all phases and areas of system development. For example, technical terms were intentionally avoided in the first few steps of the Interactive Selection Tool in order to allow nontechnical users to investigate potential geoconstruction technologies for different types of transportation applications. All users should acknowledge that a geotechnical information system deals with subject matter of realistic complexity and requires a considerable amount of human experience (Jackson 1999).

Elimination of Technologies

The process of the elimination of technologies is best described as a heuristic process. Intuition, experience, and judgment can be utilized to develop heuristic rules (Ignizio 1991). Chameau and Santamarina (1989) found that a geotechnical expert's comprehension of a problem is affected by a large number of factors, including those that are case-specific, context-dependent, and subjective. Geotechnical experts make decisions based upon the recollection of previous cases, which is very relevant in geotechnical engineering, where an emphasis is placed on experience (Chameau and Santamarina 1989).

A technically acceptable solution(s) is generally sought, rather than the optimal solution. A general characteristic of many heuristic programs is the focus on screening, filtering, or pruning to reduce the number of alternatives that are considered (Ignizio 1991). Thus, even though it is possible that a better solution might be missed, the apparently less attractive solutions are eliminated in the selection process (Ignizio 1991). The crux of viewing the selection assistance as a heuristic program is that the solution(s) identified as a result of utilizing the selection system may or may not be the best or optimal solution. The best or optimal solution requires consideration of both technical and nontechnical project issues and constraints as part of a project-specific determination completed by an experienced engineer.

Many decisions in the transportation sector are cost driven. The elimination of technologies specifically based solely on cost was avoided. A valid comparison of technology costs can only be completed after preliminary designs for multiple geoconstruction technologies have been developed. The information system provides the user with the tools to complete a preliminary design and subsequent cost analysis that captures the technology-specific costs of implementation and construction. A caveat in this respect is that within the Interactive Selection Tool a query relates to project size, and technologies were eliminated if they were not cost-effective to certain sized projects.

The Knowledge

Formalization of the expert knowledge into a usable, organized platform required significant effort and revision throughout development. "The most important process in a knowledge-based system is knowledge acquisition. How the knowledge is obtained and where it is obtained determines the usefulness of the system" (Fredlund *et al.* 1996). Two sources were utilized to acquire the knowledge for the selection system. First, knowledge for identifying potentially applicable technologies to a set of geotechnical and loading conditions came from the results of the research team's work products, and, second, from experts on the research team and project advisory board. The experts included in development of the knowledge base are shown in Supplementary Material 1. (The Supplementary Material is included in this chapter following the references.)

The research team's work efforts included the development of Comprehensive Technology Summaries, Design Method Assessments, and QC/QA Procedure Assessments for each of the 46 technologies. Development of the summaries entailed an in-depth technology overview that included advantages, potential disadvantages, applicable soil types, depth/height limits, groundwater conditions, material properties, project-specific constraints, equipment needs, and environmental considerations. The development of these summaries and assessment documents provided the initial knowledge base for each technology and the application of that technology with regard to geotechnical, loading, and project conditions.

The knowledge was then evaluated and refined by experts on the research team and advisory board during final development. Unanimous agreement among the experts did not always occur, and the knowledge base described herein represents a consensus agreement within the group.

The knowledge base refinement included heuristics, such as “Do you know a related problem,” which required the recollection of previous projects by each expert (Cheng *et al.* 2008). An important observation during development of the knowledge base is that each expert had a certain history, or unique reference set of projects and solutions, and each member preferred their set of preferences to other members’ preferences. This difficulty highlights how each person individually perceives a certain problem and develops a personally preferred solution.

Individual Technology Ratings

Technology ratings were developed to assist users in gaining a qualitative perspective on how established the use of the technology is in the U.S. and how the technology relates to the three aims of the overall research project. A qualitative assessment was completed to rate the technologies according to Degree of Technology Establishment, Potential Contribution to Rapid Renewal of Transportation Facilities, Potential Contribution to Minimal Disruption of Traffic, and Potential Contribution to Production of Long-Lived Facilities. Ratings for the technologies were determined near the end of the three-year study after development of the summaries and assessments. Based on discussion amongst the project investigators, a consensus rating for each technology for the four categories was finalized, as shown in Table 20. Ratings were implemented using Very Low, Low, Moderate, High, and Very High. Such ratings allowed the same qualitative ratings for all the categories and provided a methodology that was simpler, easier to understand, and consistent across all categories. The rating descriptions for each of the four categories are detailed in Tables 21 through 24.

Table 21. Description of ratings – degree of technology establishment

Rating		Description
Numeric	**Qualitative**	
1	Very low	The technology is not used at all in the transportation industry in the U.S.
2	Low	The technology has been used minimally in the U.S.
3	Moderate	The technology has been used moderately in the U.S.
4	High	The technology has been used on more than 30 but less than 100 transportation projects in the U.S.
5	Very high	The technology is routinely used in the transportation industry in the U.S.

Table 22. Description of ratings – potential contribution to rapid renewal of transportation facilities

Rating		Description
Numeric	**Qualitative**	
1	Very low	The technology is slower than traditionally-utilized technologies in project delivery time, but may contribute to other project objectives.
2	Low	The technology does not have the potential to be substantially different from the traditionally-utilized technologies in project delivery time.
3	Moderate	The technology has potential to be slightly faster than traditionally-utilized technologies in project delivery time.
4	High	The technology has potential to be faster than the traditionally-utilized technologies in project delivery time.
5	Very high	The technology has potential to be much faster than the traditionally-utilized technologies in project delivery time.

Table 23. Description of ratings – potential contribution to minimal disruption of traffic

Rating		Description
Numeric	**Qualitative**	
1	Very low	The technology cannot be applied without extensive and lengthy disruption of traffic 24 hours per day.
2	Low	The technology requires extensive traffic disruption 24 hours per day, but only for a short period.
3	Moderate	The technology requires minor disruption of traffic 24 hours per day for an extended period, or it requires major disruption of traffic only during times of low traffic volumes, e.g., at night.
4	High	The technology requires disruption of traffic only during times of low traffic volumes, e.g., at night, and the disruption is only minor or moderate.
5	Very high	The technology has potential to avoid all disruption of traffic.

Table 24. Description of ratings – potential contribution to production of long-lived facilities

Rating		Description
Numeric	**Qualitative**	
1	Very Low	The technology would be expected to shorten the service life of facilities compared to what is routinely achieved today.
2	Low	The technology does not have the potential to significantly affect the service lives of facilities, either positively or negatively.
3	Moderate	The technology has potential to slightly increase service lives of facilities.
4	High	The technology has potential to moderately increase service lives of facilities.
5	Very High	The technology has potential to greatly increase service lives of facilities.

Technologies by Classification

The Technology Selection component allows experienced engineers quick access to solutions according to particular classifications or categories of problems. Various categories of ground improvement technologies have been presented by many authors, as cited above in the numerous references in the Geoconstruction Technologies section. The technologies are sorted by the following eleven classifications (as detailed in Supplementary Material 2):

- Earthwork Construction
- Soft Ground Drainage and Consolidation
- Densification of Cohesionless Soils
- Construction of Vertical Support Elements
- Embankments Over Soft Soils
- Lateral Earth Support
- Cutoff Walls
- Liquefaction Mitigation
- Increased Pavement Performance
- Void Filling
- Sustainability

Interactive Selection Tool

The Interactive Selection Tool is a qualitative tool to assist the engineer in completing a project-specific, user-developed, quantitative analysis and comparison of potential technologies. The Interactive Selection Tool is a knowledge based system. Special programming forms the logic, and the knowledge is contained in a series of tables within the database. Each selection queries a database column and utilizes a nested if…then statement to sort the appropriate technologies. Only technologies that satisfy all of the queries remain as potentially applicable technologies. A significant benefit of the rule-based approach is the sharing of knowledge, especially when the knowledge is not the type of knowledge typically published in scholarly publications (Spring *et al.* 1991). Fuzzy logic and probability theory were considered for use in the development of the interactive selection system. However, a simpler rule-based system was chosen to allow the knowledge utilized to be transparent and the system to be used by a wide range of technical and nontechnical users.

At the outset of development, a list of potential queries was generated to provide an indication of all the factors that influence the selection of a geoconstruction technology, such as project type, size, constraints, depth of improvement, detailed soil conditions, groundwater conditions, desired improvements, geologic setting, and previous experience with certain techniques. The influence of other potential non-technical queries became apparent during development and included availability of experienced contractors, available materials, and project schedule. The tool outlined does not address all of the potential factors affecting the selection of a geoconstruction technology, but the tool does establish an initial framework to assist in decision-making.

Select an Application is the first decision in the interactive tool, as illustrated in Figure 17. Queries within the tool are shown in bold italics and generally followed by the possible responses in a bullet listing. The four applications are as follows:

- Construction over Unstable Soils
- Construction over Stable or Stabilized Soils
- Geotechnical Pavement Components
- Working Platforms

Supplementary Material 3 shows which technologies could potentially be utilized for each application. Each application results in a unique set of queries to winnow the possible technologies for each application. In the automated system, pop-up help windows appear next to each query to explain the purpose or intent of the posed query and to assist the user in determining the proper selection.

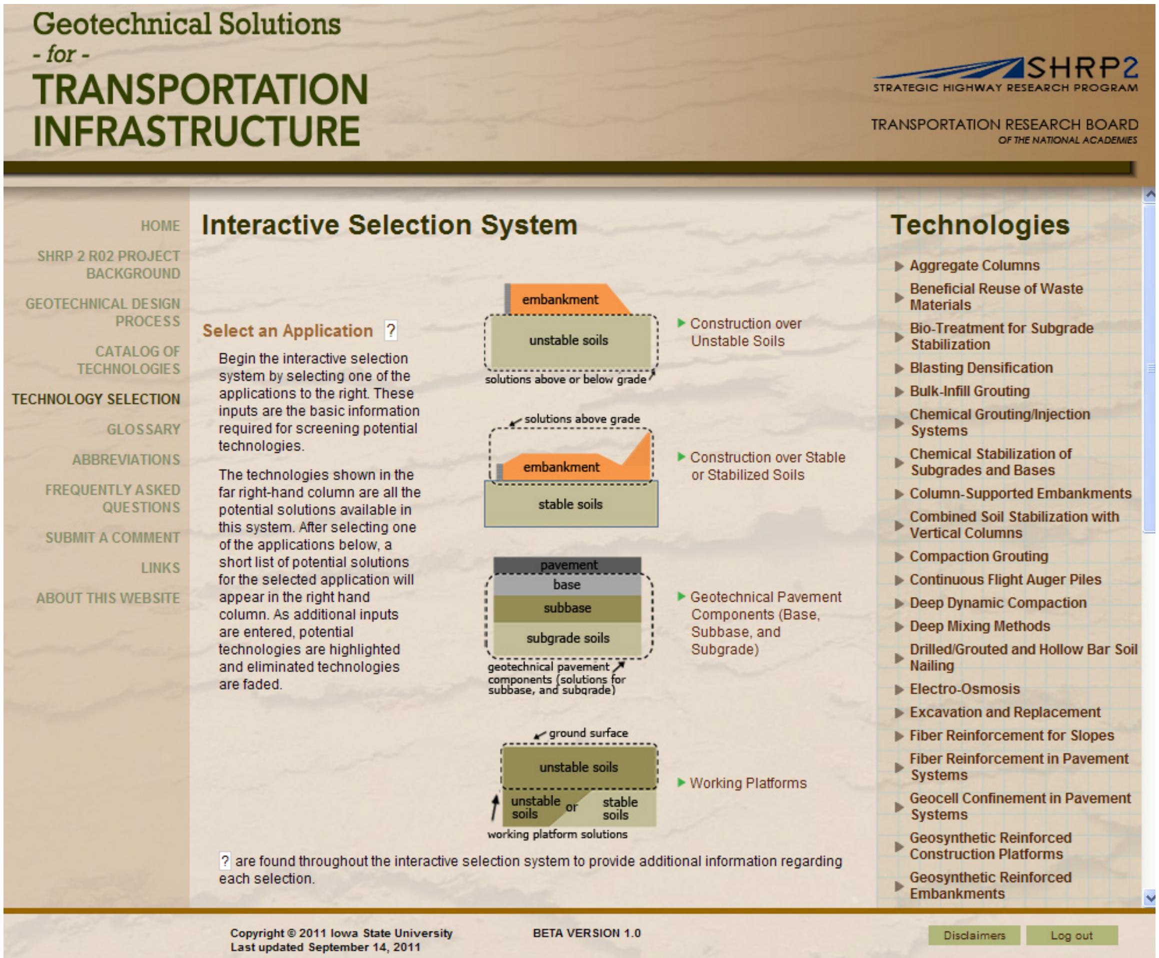

Figure 17. Interactive Selection Tool webpage

Construction over Unstable Soils

This application is focused on methods to support embankment and embankment widening on the foundation, i.e., typically below-grade technologies. Methods include ground improvement and support over the unstable soils. Although the ground improvement is often below-grade, some at-grade technologies are also applicable to this application. A flow chart depicting a portion of the decision process is shown in Figure 18.

The next query is ***Select the soil type which best describes the unstable soil condition***. Only one soil type can be selected at a time. If a mixture or differing layers of soil types exist, the user should complete an iteration for each unstable soil type by separately following the paths of the different soil types, then compare results and engineer for the project-specific conditions. The unstable soil conditions considered in the system are as follows:

- Wet and weak, fine grained soils
- Unsaturated, loose, granular soils
- Saturated, loose, granular soils
- Voids – sinkholes, abandoned mines, etc.
- Problem soils and site – expansive, sensitive, collapsing, dispersive, landfills, and existing fill

No detailed inputs for voids or problem soils and sites follow beyond this query. For the fine grained and granular soil conditions, the next step is to select the ***Depth below ground surface requiring treatment. This depth could be full-depth treatment of unstable soils or partial-depth treatment of unstable soils.*** Selection of treatment depth allows sorting of technologies that have typical installation depths. Technologies that do not penetrate the ground surface, such as lightweight fill and geosynthetic reinforced embankments, are not removed based on depth. Through the selection of depth, the system considers that the user has already identified unstable soil conditions that require mitigation. Inexperienced users should anticipate that the unstable soils will require full-depth treatment. Only experienced users should consider partial-depth treatment. The treatment depth ranges selected for inclusion in the system are as follows:

- 0 – 1.5 m
- 1.5 – 3 m
- 3 – 6 m
- 6 – 15 m
- Greater than 15 m

The soil type and depth of improvement represent the minimum level of detail required for an initial screening of technologies, and a break was intentionally made in the system to alert inexperienced users to the need to seek experienced counsel. Even with these two critical parameters identified, the list of potential technologies remained large. A screenshot from this point in the automated system is shown in Figure 19. A Project-Specific Selection decision matrix for this application was developed and includes the following ten queries. (The queries are shown in bold, italic font.) In the automated system, the listing of potential technologies updates with each selection, and a response to each query is not necessary. The technologies associated with the Project-Specific Selection and their sorting with the queries are detailed in Supplementary Material 4. Additional comments are provided for a few of the queries and selections for clarification.

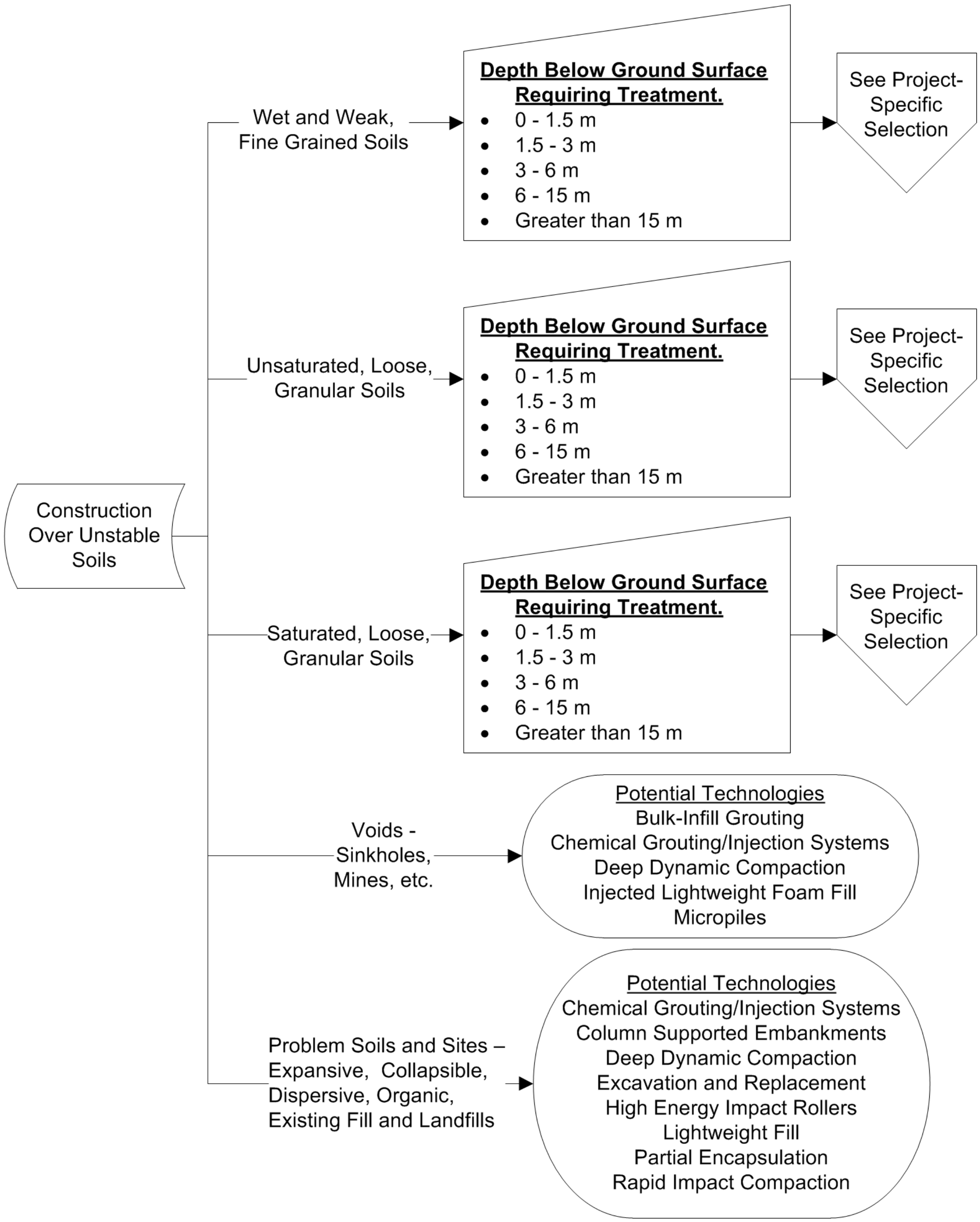

Figure 18. Portion of decision flow for construction over unstable soils

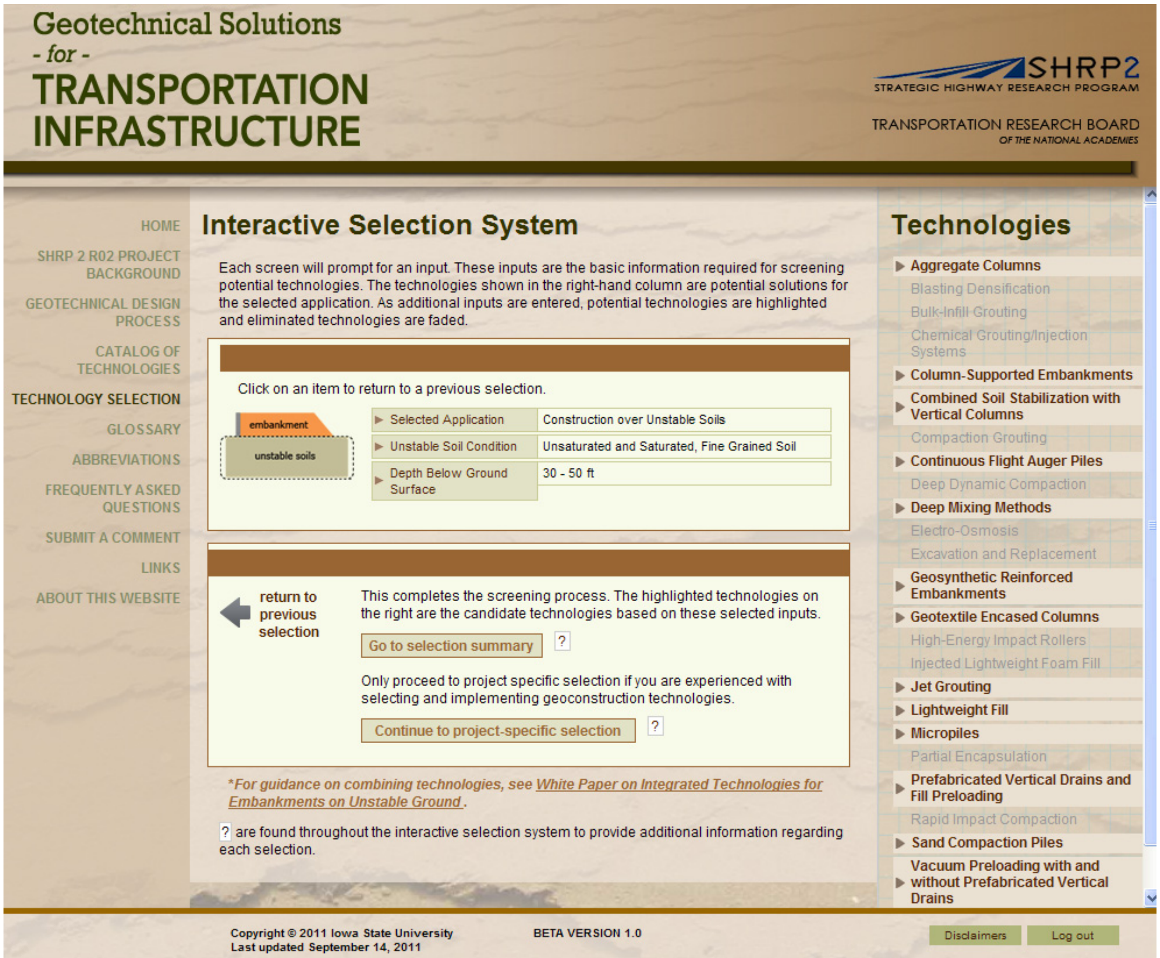

Figure 19. Sample webpage for construction over unstable soils with option to continue to project-specific selection

Select the purpose(s) of improvement for the project.

- Increase resistance to liquefaction
- Increase strength
- Increase bearing capacity
- Bypass soft ground (stiffer columns that transmit load to deeper strata)
- Reduce immediate settlement

- Seepage barrier (cutoff wall)
- Reduce consolidation settlement
- Increase rate of consolidation

Select project type. Selection of project type allows sorting of technologies based on project conditions and the usefulness of the geoconstruction technology to the specific project type.

- New embankment/new construction
- Embankment widening
- Replacement structure (any structure that has deteriorated or inadequate capacity and replacement is less expensive or disruptive to existing system)
- New structure (culvert, wall, etc.)
- Existing structure with differential settlement to be remediated

Site characteristics. Selection of site characteristics allows for a distinction in site requirements to identify viable geoconstruction technologies.

- Large, open, undeveloped sites (rural areas with plenty of working room)
- Constrained, developed sites (urban areas with limited working room)

Size of area to be improved. Selection of the size of the area to be improved allows for sorting of geoconstruction technologies because some technologies are more economical and practical for small areas while other technologies are more economical and practical for larger areas.

- Relatively small, localized area, such as under a box culvert
- Moderately sized area, such as a bridge approach embankment

- Large area, such as a significant reach of alignment

Project constraint(s). If required, selection of a project constraint sorts technologies that can typically be utilized with that constraint.

- Low overhead clearance
- Adjacent structures (such as buildings or retaining walls)
- Existing utilities

Select the best description of the construction or implementation schedule. Through selection of the best description of the construction or implementation schedule, geoconstruction technologies can be identified for accelerated construction schedules.

- Accelerated schedule (technologies that result in completion of construction more quickly than traditional solutions)
- No schedule or time constraints

Select unstable soil condition that best describes site.

- Unstable soil extends from ground surface to depth requiring improvement (weak soils are a uniform deposit extending from the ground surface to the treatment depth)
- Unstable soil underlies a stable soil (a crust or stronger soil is located above the unstable soils)
- Unstable soils and stable soils are inter-layered to depth requiring improvement

Are sufficiently thick peat layers present that will affect construction and settlement? Loading peat soils typically results in large vertical and horizontal deformations. The presence of peat soils is often problematic for certain geoconstruction technologies. If thin layers of peat are present, the user must evaluate if the peat soils will affect construction of column technologies. A common consideration is that if the thickness of the peat layer is greater than the diameter of the ground improvement column, then performance of the geoconstruction technology will likely be affected. This selection removes geoconstruction technologies that are typically not appropriate for use in or over peats.

If unstable fine grained soils are present, do the unstable soils have a shear strength less than 25 kPa? Clayey soils with a shear strength below 25 kPa generally do not provide lateral confinement for some technologies. 25 kPa is not a hard and fast rule, but is meant to provide a commonly accepted delineation for very weak soils. If the soil is layered and varies in strength, the user will have to decide if the very weak layers are sufficiently thick to affect performance. As stated with peats, a common consideration is that if the thickness of the very soft layer is greater than the diameter of the ground improvement column, then performance will be affected. This selection removes geoconstruction technologies that are typically not appropriate for use in very soft and soft, fine grained soils.

Are any subsurface obstructions present which would cause drilling difficulty, such as cobbles, boulders, buried tree trunks, or construction debris? This selection removes geoconstruction technologies that have difficulties penetrating subsurface obstructions.

Construction over Stable or Stabilized Soils

This application focuses on methods for embankment and/or embankment widening construction, i.e., above-grade technologies. Methods include fill placement and compaction procedures, reduction of embankment width/volume, fill earth retention systems, and slope stabilization systems. The ground improvement methods strengthen the embankment materials, allow for geometric constraints such as retaining walls, or stabilize slopes. The decision framework for this application is illustrated in Figure 20. The three aspects for this application are as follows:

- Enhance compaction process (both traditional and emerging technologies that are relevant to compaction of highway materials)
- Slope stabilization and earth retention (technologies for grade separation structures or technologies to stabilize existing, cut, or cut/fill slopes)
- Use of alternative or recycled materials (technologies that incorporate alternative, recycled, or waste materials in the construction, rehabilitation, or reconstruction of roadways)

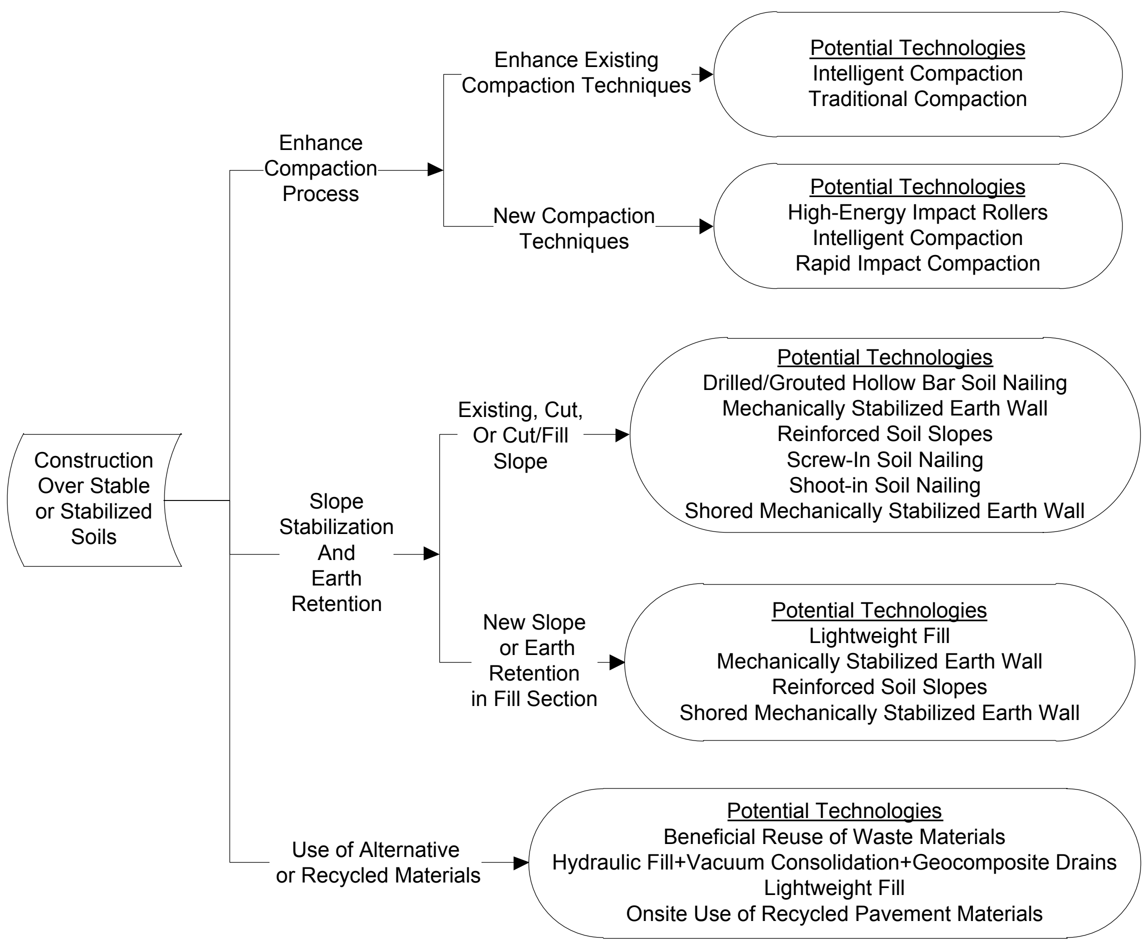

Figure 20. Portion of decision flow for construction over stable or stabilized soils

Geotechnical Pavement Components (Base, Subbase, and Subgrade)

This application focuses on methods to improve pavement construction. Methods include fill placement, stabilization, grouting, and reinforcement technologies. Recycling/reuse of materials in the pavement section is also included. A portion of the decision flow for this application is shown in Figure 21. This application requires the user to ***Select the purpose of technology application***:

- Stabilization of pavement support layer(s)
- Void filling
- Use of alternative or recycled materials in pavement support layer(s)

The void filling and alternative or recycled materials quickly result in a short list of technologies. Additional questions were developed under the stabilization of pavement support layers option. The user must next ***Select the pavement support layers to be improved***:

- Base/subbase layer
- Subgrade layer
- Deeper subgrade treatment
- In-situ treatment with the pavement surface in place

If base/subbase or subgrade layer is selected, the next step is to ***Select the Base/subbase or subgrade soil type***. The typical soil types based on the Unified Soil Classification System (USCS) and American Association of State Highway and Transportation Officials (AASTHO) are shown in Figure 21 for reference. Only the possible soil types associated with a particular pavement layer appear in the automated system. After selection of the material type, the user must ***Select the property for improvement,*** as also shown in Figure 21. Details regarding applicability of specific technologies to these selections are presented in Supplementary Material 5.

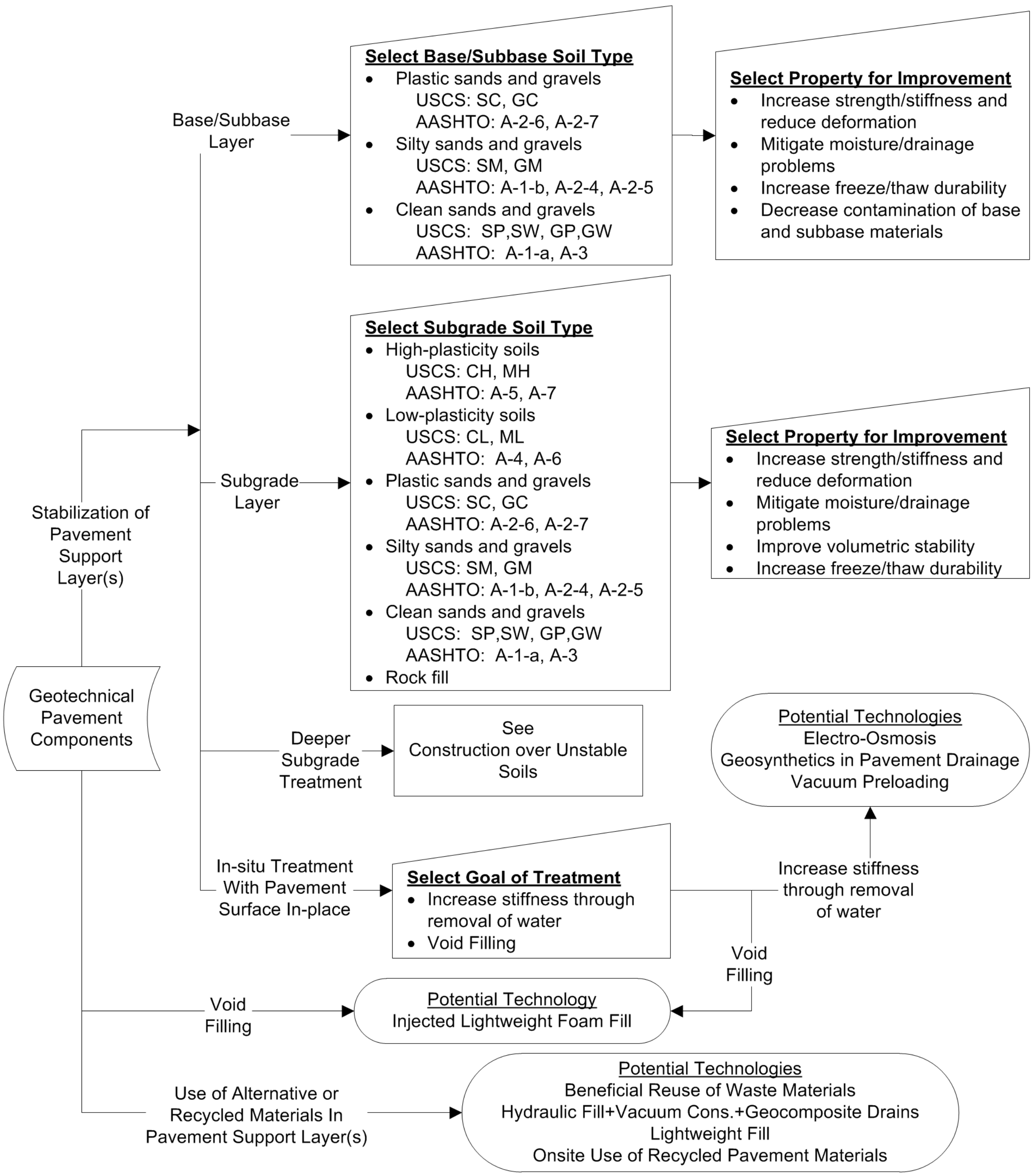

Figure 21. Portion of decision flow for geotechnical pavement components

Working Platforms

This application focuses on methods to provide temporary working platforms. Methods include fill placement, stabilization, and reinforcement technologies. Working platforms are also applicable to Construction over Unstable Soils and Geotechnical Pavement Components. The technologies include Chemical Stabilization, Excavation and Replacement, Geosynthetic Reinforced Construction Platforms, and Mechanical Stabilization. The Geotechnical Pavement Components application is the appropriate option if the working platform is to provide long-term support of the pavement layers and be considered in pavement design.

Observations from Knowledge Base Development

Several queries established at the outset of development resulted in no significant sorting of technologies after the experts refined the knowledge base. Identical potential technologies were identified for both unsaturated and saturated, loose granular soils. Similarly, the depth to groundwater and the presence of flowing sands did not eliminate any technologies. For Geotechnical Pavement Components, intuitively one would expect whether the project was new construction or rehabilitation to provide some sorting of technologies, but this distinction resulted in no significant sorting of potential technologies.

Discussion

The selection assistance helps the user in identifying and sorting possible alternatives or geoconstruction technologies. The comparison and final selection of the geoconstruction technology(s) require the engineering judgment of an experienced engineer on a project by project case. Proper application of a geoconstruction technology requires extensive background knowledge of available ground treatment technologies and careful evaluation of several factors. These factors include understanding the functions of the method, utilization of several selection criteria, the use of appropriate design procedures, implementation of the right technologies for QA/QC, and consideration of all relevant cost components and environmental factors. The technical information provided in the information system combined with the engineering judgment of the user will result in transportation projects that are built faster, cost less, and last longer.

Testing

Alpha testing has been completed by the research team members and involved both static and dynamic analysis. Static analysis involved the examination of the documents contained in the catalog of technologies and review of the knowledge base and logic behind the system. Dynamic analysis involved the actual program execution to identify and examine program failures (Naik and Tripathy 2008). All the problems identified during Alpha testing have been addressed and/or corrected.

Limitations

A limitation of the selection assistance is that the user is led to individual geoconstruction technologies, where combinations of technologies may be utilized on some projects. Additionally, other technically viable solutions may likely exist for a project beyond the list of geoconstruction technologies developed through the selection system. To address these limitations, a document is available in the automated system to provide assistance in combining technologies.

Future Enhancements

The addition of technologies, such as shallow foundations, deep foundations, and bridge abutment wall systems, would allow the system to be beneficial to practicing engineers on a routine basis. Additional advanced selection tools, similar to the Project-Specific Selection matrix, could be developed to further refine the selection of technologies for other applications. The knowledge base and programming utilized were developed with the intention of allowing future updates to the system.

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Supplementary Material 1. Principal investigators and advisory board members

Principal Investigators	
Name	**Affiliation**
Ryan Berg	Ryan R. Berg & Associates
Donald Bruce	Geosystems, L.P.
Barry Christopher	Consultant
Jim Collin	The Collin Group, Ltd.
Gary Fick	Trinity Construction
George Filz	Virginia Tech
Jie Han	University of Kansas
Jim Mitchell	Virginia Tech
Vern Schaefer	Iowa State University
Dennis Turner	Consultant
Linbing Wang	Virginia Tech
David White	Iowa State University

Advisory Board Members	
Name	**Affiliation**
James Brennan	Kansas DOT
Allen Cadden	Schnabel Engineering, North
Mike Cowell	GeoConstructors, Inc.
David Horhota	Florida DOT
Mark Morvant	Louisiana TRC
Hooshmand Nikoui	Caltrans
Seth Pearlman	DGI-Menards, Inc.
Steve Saye	Kiewit Engineering
Al Sehn	Hayward Baker, Inc.
David Shiells	Virginia DOT
John Siekmeier	Minnesota DOT

Supplementary Material 2. Technologies by classification

Technology	Earthwork construction	Soft ground drainage & consolidation	Densification of cohesionless soils	Construction of vertical support elements	Embankments over soft soils	Lateral earth support	Cutoff walls	Liquefaction mitigation	Increased pavement performance	Void filling	Sustaina-bility
Aggregate columns			✓	✓				✓			
Beneficial reuse of waste materials											✓
Bio-treatment for subgrade stabilization	✓										
Blasting densification			✓					✓			
Bulk-infill grouting										✓	
Chemical grouting/ injection systems										✓	
Chemical stabilization of subgrades and bases	✓										
Column-supported embankments					✓						
Combined soil stabilization with vertical columns				✓							
Compaction grouting			✓					✓			
Continuous flight auger piles				✓							
Deep dynamic compaction			✓					✓			
Deep mixing methods				✓			✓	✓			
Drilled/grouted and hollow bar soil nailing						✓					
Electro-osmosis		✓									
Excavation and replacement	✓							✓			
Fiber reinforcement in pavement systems									✓		
Geocell confinement in pavement systems	✓								✓		

Supplementary Material 2. (continued)

Technology	Earthwork construction	Soft ground drainage & consolidation	Densification of cohesionless soils	Construction of vertical support elements	Embankments over soft soils	Lateral earth support	Cutoff walls	Liquefaction mitigation	Increased pavement performance	Void filling	Sustaina-bility
Geosynthetic reinforced construction platforms	✓										
Geosynthetic reinforced embankments					✓						
Geosynthetic reinforcement in pavement systems									✓		
Geosynthetic separation in pavement systems									✓		
Geosynthetics in pavement drainage									✓		
Geotextile encased columns				✓							
High-energy impact rollers	✓		✓								
Hydraulic fill with geocomposite and vacuum consolidation		✓									✓
Injected lightweight foam fill										✓	
Intelligent compaction	✓		✓								
Jet grouting				✓			✓	✓			
Lightweight fills					✓						
Mechanical stabilization of subgrades and bases	✓										
Mechanically stabilized earth wall systems						✓					
Micro-piles				✓							
Onsite use of recycled pavement materials											✓

Supplementary Material 2. (continued)

Technology	Earthwork construction	Soft ground drainage & consolidation	Densification of cohesionless soils	Construction of vertical support elements	Embankments over soft soils	Lateral earth support	Cutoff walls	Liquefaction mitigation	Increased pavement performance	Void filling	Sustaina-bility
Partial encapsulation									✓		
PVDs and fill preloading		✓									
Rapid impact compaction	✓		✓								
Reinforced soil slopes						✓					
Sand compaction piles			✓	✓				✓			
Screw-in soil nailing						✓					
Shoot-in soil nailing						✓					
Shored mechanically stabilized earth wall system						✓					
Traditional compaction	✓										
Vacuum preloading with and without PVDs		✓									
Vibrocompaction			✓					✓			
Vibro-concrete columns				✓							

Supplementary Material 3. Geoconstruction technology application

Technology	Application			
	Construction over unstable soils	Construction over stable or stabilized soils	Geotechnical pavement components	Working platforms
Aggregate columns	✓			
Beneficial reuse of waste materials		✓	✓	
Bio-treatment for subgrade stabilization			✓	
Blasting densification	✓			
Bulk-infill grouting	✓		✓	
Chemical grouting/ injection systems	✓		✓	
Chemical stabilization of subgrades and bases			✓	✓
Column-supported embankments	✓			
Combined soil stabilization with vertical columns	✓			
Compaction grouting	✓			
Continuous flight auger piles	✓			
Deep dynamic compaction	✓			
Deep mixing methods	✓			
Drilled/grouted and hollow bar soil nailing		✓		
Electro-osmosis	✓		✓	
Excavation and replacement	✓		✓	✓
Fiber reinforcement in pavement systems			✓	
Geocell confinement in pavement systems			✓	
Geosynthetic reinforced construction platforms				✓
Geosynthetic reinforced embankments	✓			
Geosynthetic reinforcement in pavement systems			✓	
Geosynthetic separation in pavement systems			✓	
Geosynthetics in pavement drainage			✓	

Supplementary Material 3. (continued)

Technology	Application			
	Construction over unstable soils	Construction over stable or stabilized soils	Geotechnical pavement components	Working platforms
Geotextile encased columns	✓			
High-energy impact rollers	✓	✓		
Hydraulic fill with geocomposite and vacuum consolidation		✓	✓	
Injected lightweight foam fill	✓		✓	
Intelligent compaction		✓	✓	
Jet grouting	✓			
Lightweight fills	✓	✓	✓	
Mechanical stabilization of subgrades and bases			✓	✓
Mechanically stabilized earth wall systems		✓		
Micro-piles	✓			
Onsite use of recycled pavement materials		✓	✓	
Partial encapsulation	✓		✓	
PVDs and fill preloading	✓			
Rapid impact compaction	✓	✓		
Reinforced soil slopes		✓		
Sand compaction piles	✓			
Screw-in soil nailing		✓		
Shoot-in soil nailing		✓		
Shored mechanically stabilized earth wall system		✓		
Traditional compaction		✓	✓	
Vacuum preloading with and without PVDs	✓		✓	
Vibrocompaction	✓			
Vibro-concrete columns	✓			

Supplementary Material 4. Project-specific selection decision matrix (Part 1 of 5)

Technology	Soil type			Treatment depth				
	Wet and weak fine grained soils	Unsaturated, loose granular soils	Saturated, loose granular soils	0 – 1.5 m	1.5 – 3 m	3 – 6 m	6 – 15 m	Greater than 15 m
Aggregate columns	✓	✓	✓		✓	✓	✓	✓
Blasting densification		✓	✓			✓	✓	✓
Chemical grouting/injection systems		✓	✓		✓	✓	✓	✓
Column-supported embankments	✓	✓	✓		✓	✓	✓	✓
Combined soil stabilization with vertical columns	✓	✓	✓		✓	✓	✓	
Compaction grouting		✓	✓		✓	✓	✓	✓
Continuous flight auger piles	✓	✓	✓			✓	✓	✓
Deep dynamic compaction		✓	✓			✓		
Deep mixing methods	✓	✓	✓			✓	✓	✓
Electro-osmosis	✓			✓	✓	✓		
Excavation and replacement	✓	✓	✓	✓	✓			
Geosynthetic reinforced Embankments	✓			✓	✓	✓	✓	✓
Geotextile encased columns	✓				✓	✓	✓	
High-energy impact rollers		✓	✓	✓				
Jet grouting	✓	✓	✓			✓	✓	✓
Lightweight fill	✓			✓	✓	✓	✓	✓
Micropiles	✓	✓	✓			✓	✓	✓
PVDs and fill preloading	✓					✓	✓	✓
Rapid impact compaction		✓	✓	✓	✓	✓		
Sand compaction piles	✓	✓	✓			✓	✓	✓
Vacuum preloading with and without prefabricated vertical drains	✓				✓	✓	✓	✓
Vibrocompaction		✓	✓			✓	✓	✓
Vibro-concrete columns	✓	✓	✓			✓	✓	✓

Supplementary Material 4. (continued)

Technology	Purpose of improvement							
	Increase resistance to liquefaction	Increase strength	Increase bearing capacity	Bypass soft ground	Reduce immediate settlement	Seepage barrier (cutoff wall)	Reduce consolidation settlement	Increase rate of consolidation
Aggregate columns	✓	✓	✓		✓		✓	✓
Blasting densification	✓	✓	✓		✓			
Chemical grouting/injection systems	✓	✓	✓		✓	✓		
Column-supported embankments			✓	✓	✓		✓	
Combined soil stabilization with vertical columns	✓	✓	✓	✓	✓		✓	
Compaction grouting	✓	✓	✓		✓			
Continuous flight auger piles			✓	✓	✓		✓	
Deep dynamic compaction	✓	✓	✓		✓			
Deep mixing methods	✓	✓	✓	✓	✓	✓	✓	
Electro-osmosis		✓	✓		✓		✓	✓
Excavation and replacement	✓	✓	✓		✓		✓	
Geosynthetic reinforced Embankments								
Geotextile encased columns			✓	✓			✓	✓
High-energy impact rollers	✓	✓	✓		✓			
Jet grouting	✓	✓	✓	✓	✓	✓	✓	
Lightweight fill					✓		✓	
Micropiles			✓	✓	✓		✓	
PVDs and fill preloading		✓	✓				✓	✓
Rapid impact compaction	✓	✓	✓		✓			
Sand compaction piles	✓	✓	✓		✓		✓	✓
Vacuum preloading with and without PVDs		✓	✓				✓	✓
Vibrocompaction	✓	✓	✓		✓			
Vibro-concrete columns			✓	✓	✓		✓	

Supplementary Material 4. (continued)

Technology	Project type					Site characteristics	
	New embankment/ new construction	Embankment widening	Replacement structure	New Structure (culvert, wall, etc.)	Existing Structure with Differential Settlement	Large, open, undeveloped sites	Constrained, developed sites
Aggregate columns	✓	✓	✓	✓		✓	✓
Blasting densification	✓			✓		✓	
Chemical grouting/injection systems			✓	✓	✓		✓
Column-supported embankments	✓	✓	✓	✓		✓	✓
Combined soil stabilization with vertical columns	✓	✓	✓	✓		✓	✓
Compaction grouting	✓	✓	✓	✓	✓		✓
Continuous flight auger piles	✓	✓	✓	✓		✓	✓
Deep dynamic compaction	✓			✓		✓	
Deep mixing methods	✓	✓	✓	✓		✓	✓
Electro-osmosis	✓	✓	✓	✓	✓	✓	✓
Excavation and replacement	✓	✓	✓	✓		✓	✓
Geosynthetic reinforced Embankments	✓	✓		✓		✓	
Geotextile encased columns	✓	✓				✓	✓
High-energy impact rollers	✓	✓	✓	✓		✓	
Jet grouting	✓	✓	✓	✓	✓		✓
Lightweight fill	✓	✓				✓	✓
Micropiles		✓	✓	✓	✓		✓
PVDs and fill preloading	✓	✓		✓		✓	
Rapid impact compaction	✓	✓	✓	✓		✓	✓
Sand compaction piles	✓	✓	✓	✓		✓	✓
Vacuum preloading with and without PVDs	✓	✓		✓		✓	✓
Vibrocompaction	✓	✓		✓		✓	
Vibro-concrete columns	✓	✓	✓	✓		✓	✓

Supplementary Material 4. (continued)

Technology	Size of area to be improved			Project constraints			Construction schedule	
	Small area	Moderate area	Large area	Low overhead clearance	Adjacent buildings	Existing utilities	Accelerated schedule	No schedule constraints
Aggregate columns	✓	✓	✓		✓	✓	✓	✓
Blasting densification		✓	✓				✓	✓
Chemical grouting/injection systems	✓			✓	✓	✓	✓	✓
Column-supported embankments	✓	✓	✓	✓	✓	✓	✓	✓
Combined soil stabilization with vertical columns	✓	✓	✓		✓	✓	✓	✓
Compaction grouting	✓	✓		✓	✓	✓	✓	✓
Continuous flight auger piles	✓	✓	✓		✓	✓	✓	✓
Deep dynamic compaction		✓	✓				✓	✓
Deep mixing methods		✓	✓		✓	✓	✓	✓
Electro-osmosis	✓	✓		✓	✓			✓
Excavation and replacement	✓	✓	✓	✓	✓	✓	✓	✓
Geosynthetic reinforced Embankments	✓	✓	✓	✓				✓
Geotextile encased columns	✓	✓	✓				✓	✓
High-energy impact rollers	✓	✓	✓	✓			✓	✓
Jet grouting	✓	✓		✓	✓	✓	✓	✓
Lightweight fill	✓	✓	✓	✓	✓	✓	✓	✓
Micropiles	✓	✓		✓	✓	✓	✓	✓
PVDs and fill preloading	✓	✓	✓					✓
Rapid impact compaction	✓	✓	✓	✓			✓	✓
Sand compaction piles	✓	✓	✓		✓	✓	✓	✓
Vacuum preloading with and without PVDs	✓	✓	✓	✓				✓
Vibrocompaction	✓	✓	✓				✓	✓
Vibro-concrete columns	✓	✓	✓		✓	✓	✓	✓

Supplementary Material 4. (continued)

Technology	Unstable soil condition that best describes site			Sufficiently thick peat layers are present that will affect construction and settlement[a]	Fine grained soils are present with shear strengths below 25 kPa[a]	Obstructions are present which would cause drilling difficulty[a]
	Unstable soil extends from ground surface to depth	Unstable soil underlies stable soil	Unstable soils and stable soils are interlayered			
Aggregate columns	✓	✓	✓			
Blasting densification	✓	✓				✓
Chemical grouting/injection systems	✓	✓	✓		✓	✓
Column-supported embankments	✓	✓	✓	✓	✓	
Combined soil stabilization with vertical columns	✓	✓	✓		✓	
Compaction grouting	✓	✓	✓			✓
Continuous flight auger piles	✓	✓	✓	✓	✓	
Deep dynamic compaction	✓					✓
Deep mixing methods	✓	✓	✓	✓	✓	
Electro-osmosis	✓	✓	✓		✓	
Excavation and replacement	✓		✓	✓	✓	✓
Geosynthetic reinforced Embankments	✓	✓	✓	✓	✓	✓
Geotextile encased columns	✓	✓	✓	✓	✓	
High-energy impact rollers	✓					✓
Jet grouting	✓	✓	✓	✓	✓	
Lightweight fill	✓	✓	✓	✓	✓	✓
Micropiles	✓	✓	✓	✓	✓	✓
PVDs and fill preloading	✓	✓	✓	✓	✓	
Rapid impact compaction	✓					✓
Sand compaction piles	✓	✓	✓			
Vacuum preloading with and without PVDs	✓	✓	✓	✓	✓	
Vibrocompaction	✓	✓	✓			
Vibro-concrete columns	✓	✓	✓	✓	✓	

[a] All technologies are applicable if the condition is not present.

Supplementary Material 5. Geotechnical pavement components selection decision matrix

Base/subbase layer

Technology	Plastic sands and gravels				Silty sands and gravels				Clean sands and gravels	
	Increase strength/ stiffness	Mitigate moisture/ drainage problems	Increase freeze/ thaw durability	Decrease contami-nation	Increase strength/ stiffness	Mitigate moisture/ drainage problems	Increase freeze/ thaw durability	Decrease contami-nation	Increase strength/ stiffness	Decrease contami-nation
Bio-treatment for subgrade stabilization										
Chemical stabilization of subgrades and bases	✓	✓	✓		✓	✓	✓		✓	
Electro-osmosis										
Excavation and replacement	✓	✓	✓		✓	✓	✓			
Fiber reinforcement in pavement systems	✓				✓				✓	
Geocell confinement in pavement systems	✓				✓				✓	
Geosynthetic reinforcement in pavement systems	✓				✓				✓	
Geosynthetic separation in pavement systems				✓				✓		✓
Geosynthetics in pavement drainage		✓				✓				
Intelligent compaction	✓				✓				✓	
Mechanical stabilization of subgrades and bases	✓				✓				✓	
Partial encapsulation										
Traditional compaction	✓				✓				✓	

Supplementary Material 5. (continued)

Subgrade layer (Part 1 of 2)

Technology	High-plasticity soils			Low-plasticity soils		
	Increase strength/ stiffness	**Mitigate moisture/ drainage problems**	**Improve volumetric stability**	**Increase strength/ stiffness**	**Mitigate moisture/ drainage problems**	**Increase freeze/ thaw durability**
Bio-treatment for subgrade stabilization	✓			✓		
Chemical stabilization of subgrades and bases	✓	✓	✓	✓	✓	✓
Electro-osmosis	✓	✓		✓	✓	
Excavation and replacement	✓	✓	✓	✓	✓	✓
Fiber reinforcement in pavement systems				✓		
Geocell confinement in pavement systems	✓			✓		
Geosynthetic reinforcement in pavement systems	✓			✓		
Geosynthetic separation in pavement systems						
Geosynthetics in pavement drainage		✓			✓	
Intelligent compaction	✓			✓		
Mechanical stabilization of subgrades and bases	✓			✓		
Partial encapsulation			✓			
Traditional compaction	✓			✓		

Supplementary Material 5. (continued)

Subgrade layer (Part 2 of 2)

Technology	Plastic sands and gravels			Silty sands and gravels			Clean sands and gravels	Rock fill	
	Increase strength/ stiffness	Mitigate moisture/ drainage problems	Increase freeze/ thaw durability	Increase strength/ stiffness	Mitigate moisture/ drainage problems	Increase freeze/ thaw durability	Increase strength/ stiffness	Increase strength/ stiffness	Mitigate moisture/ drainage problems
Bio-treatment for subgrade stabilization	✓			✓			✓		
Chemical stabilization of subgrades and bases	✓	✓	✓	✓	✓	✓	✓		
Electro-osmosis									
Excavation and replacement	✓	✓	✓	✓	✓	✓		✓	✓
Fiber reinforcement in pavement systems	✓			✓			✓		
Geocell confinement in pavement systems	✓			✓			✓		
Geosynthetic reinforcement in pavement systems	✓			✓			✓	✓	
Geosynthetic separation in pavement systems									
Geosynthetics in pavement drainage		✓			✓				✓
Intelligent compaction	✓			✓			✓	✓	
Mechanical stabilization of subgrades and bases	✓			✓			✓	✓	
Partial encapsulation									
Traditional compaction	✓			✓			✓	✓	

CHAPTER 5. RELIABILITY OF ESTIMATING SETTLEMENTS FOR STONE COLUMNS

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S. Caleb Douglas[13 14], P.E. and Vernon R. Schaefer[15 16], P.E.

Abstract

Many estimating methods have been developed for predicting the settlement of ground reinforced by stone columns. The Priebe method is the most common method utilized in practice. Even though the Priebe method does not capture all the parameters that affect the performance of stone column reinforced ground, the method is preferred due to its simplicity. An extensive literature search provided data to evaluate the Priebe method. The concept of reliability was incorporated to help analyze the method. The Priebe method was found to have an approximately 89% probability that the measured settlement will be smaller than the

[13] Ph.D. candidate, Civil Engineering, Iowa State University, Ames, IA 50010-3232. E-mail: calebd@iastate.edu
[14] Primary researcher and author, author for correspondence
[15] Professor of Civil Engineering, Iowa State University, Ames, IA 50010-3232. E-mail: vern@iastate.edu
[16] Secondary author

estimated settlement. The Priebe method is not always conservative, and settlements may exceed those estimated.

Introduction

Vibratory ground improvement methods originated in Germany in the 1930s. Initially, the vibratory methods were used to densify clean, granular materials at depth, which was termed vibro-compaction. To improve weak, cohesive soils, the vibro-replacement technique to construct stone columns was developed using identical equipment to vibro-compaction. Stone columns have also been used in granular soils and fill materials. Stone column development, including both the vibro-replacement and vibro-displacement techniques, has been well described by Barksdale and Bachus (1983), Charles and Watts (2002), Elias *et al.* (2006), and Kirsch and Kirsch (2010). McCabe *et al.* (2009) provided a recent description of the construction methods for stone columns.

The advantages of stone columns include increasing bearing capacity, increasing global stability, and decreasing settlements. Design of stone columns is typically done by iteratively determining the most economical layout pattern of columns (triangular or rectangular), spacing, and depth of stone columns to meet project requirements. A common purpose of using stone columns is to satisfy project requirements with regard to settlement, and the settlement-specific analysis typically governs the final stone column configuration. Bearing capacity and global stability analyses require ultimate limit state design approaches with partial factors based on Eurocode 7 or a factor of safety approach in the U.S. Whereas ultimate limit state analyses include factor of safety or reliability considerations, settlements are explicitly computed using a serviceability limit state design approach with working loads and measured/estimated soil properties. No allowances for variability in the loads or resistances are considered in settlement analyses.

The literature over the last two decades is clear on the inadequacies of present settlement prediction methods for stone column reinforced ground. For example, Allen *et al.* (1991) stated that "improvements in the semi-empirical settlement prediction methods involving stone columns are needed." Clemente and Davie (2000) found that "the results from full-scale tests show more improvement than predicted by theoretical procedures, although a large scatter was observed." Abdrabbo and Mahmoud (2002) stated that "there is no reliable procedure for settlement calculation of improved geomaterial by stone columns." Raman (2006) found measured settlements in stone column reinforced areas to be 42% of the predicted settlements. The lack of a validated design procedure for estimating settlements is best illustrated by the results of a settlement prediction exercise for an embankment built on weak fine grained soils summarized by Mestat *et al.* (2006). Seventeen participants submitted settlement predictions using the finite element method, the discrete element method, the Priebe method, and other methods. Mestat *et al.* (2006) concluded that the exercise showed that "the calculation of settlement of improved soil by stone columns is complicated and remains a problem for practical applications." McCabe *et al.* (2009) found that confident predictions of settlement performance for stone column reinforced ground were problematic.

Over 250 literature records were collected that specifically address stone columns. Existing methods of estimating settlements for stone columns were identified, and case histories with measured settlements were reviewed. Although many methods of estimating settlements exist, the Priebe method is the focus of this paper. McCabe *et al.* (2009) stated that the Priebe method of estimating settlements is at present the most favored design approach of leading stone column designers. Greenwood and Kirsch (1984) concluded that the simplicity of the Priebe method applying an improvement factor to conventional calculations is attractive to engineers, which results in the method being widely used. The improvement factor is defined as the unreinforced settlement divided by the reinforced settlement. The case histories identified provide the data to allow a comparison of estimated and measured settlement for both unreinforced ground and reinforced ground. The estimated and measured settlements used in this evaluation were obtained from published case histories and are considered representative of common geotechnical practice. This is a key consideration to reduce bias in the evaluation because the writers did not have to make any assumptions or calculations to develop the data points.

Previous evaluations of projects that measured settlements in stone column reinforced areas compared the estimated improvement factor to the field measured improvement factor (Balaam and Poulos 1983; Meyerhof 1984; Besancon *et al.* 1984; Greenwood and Kirsch 1984; Clemente and Davie 2000; Charles and Watts 2002; Charles 2002; McCabe *et al.* 2009; Ellouze *et al.* 2010; McCabe and Egan 2010). These improvement factor evaluations resulted in two classes of data. The first class of data resulted from projects in which both unreinforced and reinforced areas were loaded, which allowed a direct computation of the settlement ratio. The second class of data resulted from projects in which the unreinforced settlement was estimated, and the settlement ratio was determined using the field-measured settlement of the reinforced area. The complete process of estimating settlements was not considered in the evaluations of the improvement factor.

Methods of Estimating Stone Column Settlements

Greenwood (1991) concluded that under widespread vertical loads ground strengthened by arrays of columns behave in complex ways. Early methods of estimating settlements of stone column reinforced ground were strictly empirical and semi-empirical. Theoretical models of the relationship between the stone columns and the in situ soil were presented in the 1970s. Since the 1970s, 17 or more design methods have been developed to estimate the settlement of stone column reinforced soil. The design methods developed have been based on elastic theory, limited field data, a combination of theory and field data, laboratory experiments, and/or numerical modeling studies. Settlement analyses of stone columns remain semi-empirical for day-to-day designs.

The Priebe method was found to be the most often cited method for estimating settlements of stone column reinforced ground. A summary of 15 published methods of estimating stone column settlements is provided in Table 25. Bouassida *et al.* (2003) and Normes Francaises (2005) are two additional methods, but these could not be located in English and are not included in Table 25. Finite element analysis is not typically performed on routine projects. The reader is referred to the following references that used computer modeling to estimate settlements: Majorana *et al.* (1983), Schweiger and Pande (1986), Ambily and Gandhi (2004), Clemente *et al.* (2005), Tan and Oo (2005), Abdelkrim and de Buhan (2007), Elshazly *et al.* (2007), Tan *et al.* (2008), Ellouze and Bouassida (2009), Weber *et al.* (2009), Weber (2010), Zahmatkesh and Choobbasti (2010a, 2010b), and Mohamedzein and Al-Shibani (2011).

The estimation of settlements of stone column reinforced ground using the Priebe method can generally be broken down into two steps. The first step requires completion of an unreinforced settlement estimate considering the influence of the load. The second step then determines the settlement of the reinforced ground based on an improvement factor. Each step has the potential for contributing to the wide range of outcomes for reinforced ground observed in the case histories.

Table 25. Summary of methods for estimating settlements

	Method details				
Method	**Unit cell idealization (Yes/No)**	**Equal strain assumption (Yes/No)**	**Method theory**	**Untreated settlement required (Yes/No)**	**Comments**
Greenwood (1970)	No	No	No	Yes	Empirical correlation with spacing of columns and strength of clay soils.
Hughes and Withers (1974)	Yes	Yes	Plastic	Yes	Early design method for widespread loading.
Incremental method (Goughnour and Bayuk 1979a)	Yes	Yes	Elastic-Plastic	Yes	Considered load intensity in elastic-plastic behavior.
Balaam and Booker (1981 and 1985)	Yes	Yes	Elastic	Yes	Results similar to Priebe method. Considered rigid foundation.
Balaam and Poulos (1983)	Yes	Yes	Elastic-Plastic	Yes	Results similar to Priebe method. Both rigid and flexible loading.
Equilibrium (Barksdale and Bachus 1983)	Yes	Yes	None	No	Uses the SCR to determine stress reduction in soil to estimate settlements.
FEM[a] Settlement Charts (Barksdale and Bachus 1983)	Yes	Yes	Elastic-Plastic	No (requires column length)	Incorporates load dependent behavior of overall system.

Table 25. (continued)

Method	Unit cell idealization (Yes/No)	Equal strain assumption (Yes/No)	Method theory	Untreated settlement required (Yes/No)	Comments
	Method details				
Van Impe and De Beer (1983)	No Plane Strain	Yes	Elastic	Yes	Design charts to estimate settlements.
Priebe (1995)	Yes	Yes	Elastic	Yes	Considered infinitely wide reinforced area originally, modified for footings in 1995.
Chow (1996)	Yes	Yes	Yes	Yes	Simple method developed for sand compaction piles. Similar results to the Balaam and Booker.
Alamgir *et al.* (1996)	Yes	No	Elastic-Plastic	Yes	Allowed surrounding soil to settle more than stone column.
Poorooshasb and Meyerhof (1997)	Yes	Yes	Elastic-Plastic	Yes	Priebe method is special case of general equation derived for study.
Pulko and Majes (2005)	Yes	Yes	Elastic-Plastic	Yes	Considered rigid footings.
Ambily and Gandhi (2007)	Yes	Yes	Elastic-Plastic	Yes	Similar results to the Priebe method.
Borges *et al.* (2009)	Yes	Yes	Elastic-Plastic	Yes	Results in the range of Priebe method, and Balaam and Booker

[a] FEM: finite element method

The Priebe Method

Priebe initially published his design procedure in 1976 in German. Since the initial work in 1976, Priebe adapted, extended, and supplemented the design procedure, as reported in Priebe (1991), and the process culminated in the procedure set forth in Priebe (1995). Priebe (1995) provided design procedures and design charts for various aspects of stone column design, including settlement reduction, bearing capacity, shear values of improved ground, settlement of footings, and liquefaction. Priebe (1995) contrasted vibro-replacement with vibro-compaction and concluded that only considerable efforts like large-scale load tests can prove the benefit of stone columns. Priebe (1995) stated, "The design method refers to the improving effect of stone column in a soil which is otherwise unaltered in comparison to the initial state." If the installation changes the engineering properties of the soil between the columns, the soil must be evaluated before the design of vibro-replacement can be accomplished. The assumptions and procedures associated with analyzing the reduction in settlements were well documented in Priebe (1995), and the reader is referred to that paper for a description of the estimating procedure. Ellouze *et al.* (2010) provide a criticism of the Priebe method. The Priebe method results in an improvement factor based on the area replacement ratio and strength of the column material. The estimated reinforced settlement is calculated as the estimated unreinforced settlement divided by the improvement factor.

Difficulties of Estimating Settlements

As Osterberg (1986) stated, "[T]he realities of foundation engineering are that we never find actual conditions the same as we anticipated." Three areas that can potentially contribute to the wide range of estimate outcomes are (1) design parameters selection, (2) installation effects, and (3) stress distribution.

Design Parameter Selection

Poulos (2000) said in relation to unreinforced sites that "settlement predictions are far more sensitive to the geotechnical parameters and site characterization than to the method of analysis." A quality site investigation is required to properly identify the geotechnical parameters and variability of those parameters across the site. The site investigation should leave no areas of serious doubt concerning soil conditions, engineering properties, chemical properties, and groundwater conditions (Slocombe 2001). Numerical and analytical models are of limited value for settlement prediction due to the difficulty of obtaining accurate soil and stone properties (Ashmawy *et al.* 2000).

Installation Effects

Vibrated stone column installation methods have significant variation in performance as a result of the construction technique (Bell 2004; McCabe *et al.* 2009). Case histories confirm the lack of a consistent response of the in situ soils due to stone column installation (Watts *et al.* 2000; White *et al.* 2002; Kirsch 2006, 2009; Guetif *et al.* 2007; Elshazly *et al.* 2008; Egan *et al.* 2009; Kirsch and Kirsch 2010; Castro and Karstunen 2010). Specific equipment operating on a specific site using a specific installation method will result in a unique effect on the in situ soil properties post-installation. No clear, accepted means of anticipating installation effects has been identified, but what is clear is that the installation effects influence the performance of the stone columns (Egan *et al.* 2009). An advantage of the Priebe method in this respect is that the method quantifies the improvement that results from the inclusion of the stone column without any quantification of the densification of the soil between stone columns. However, the problematic installation effects of stone columns in sensitive soils have been well documented (McKenna *et al.* 1975; Wijeyakulasuriya *et al.* 1999; Gue and Tan 2003; Oh *et al.* 2007a, 2007b).

Stress Distribution

Settlement estimates typically include both the reinforced zone and the underlying unreinforced zone. An initial consideration in evaluating the stress distribution is whether the loading is rigid or flexible. Balaam and Poulos (1983) found the reduction in settlement of a flexible foundation supported by stone columns to be slightly less than that of a rigid foundation. The behavior and stress distribution of stone columns is quite different from an isolated stone column supporting a footing to a group of stone columns supporting a rigid footing to a large array of stone columns supporting an embankment (Wehr 2004, 2006). Although stress distribution is not explicitly discussed in Priebe (1995), evaluation of the stresses in the example calculations provided in Priebe (1995) indicate that a Boussinesq-type analysis was used to estimate the stresses in the unreinforced soils. Approximations of stress distributions were presented for similar aggregate column systems by Aboshi *et al.* (1979), Bowles (1982), Lawton *et al.* (1994), Fox and Cowell (1998), and Sehn and Blackburn (2008).

One of the details not identified in the literature search is to what depth settlements should be determined below an embankment or structure constructed on stone column reinforced soils. In the design of footings, Eurocode 7 allows the analysis to only consider the zone where the increase in effective stress due to increased loading is greater than 20% of the in situ effective stress (Bond and Harris 2008). Common U.S. practice is to consider the zone to where the increase in effective stress is greater than 10%. Numerical modeling does offer the benefit of providing information regarding distribution of stresses and strains (Barksdale and Bachus 1983; Ashmawy *et al.* 2000).

Evaluation of Settlement Estimates

Over 100 stone column case histories were identified during this study. Case histories where stone columns performed satisfactorily are summarized in the Supplementary Data provided in Appendix A and sorted by the following conditions: predominately fine-grained soil case histories, predominately coarse-grained soil case histories to mitigate static settlements, predominately increasing resistance to liquefaction case histories, and predominately improvement of fill/demolition debris/refuse case histories. This extensive literature review confirms the conclusion by Barksdale and Bachus (1983) over 25 years ago, and more recently by McCabe *et al.* (2009), that there is a lack of field studies that appropriately capture all the information required to develop a complete understanding of the behavior of stone column reinforced ground.

The case histories varied greatly with regard to the information provided for site conditions, soil parameters, design considerations, construction process, and settlement monitoring. The initial goal was to identify case histories that provided detailed site and design information, which would allow completion of the Priebe design method and comparison with measured settlements in the field. Even with the numerous case histories found, very few case histories provided sufficient details to allow completion of the Priebe method. Fortunately, case histories were identified that contained settlements estimated with the Priebe method and measured settlements.

Evaluation of Estimating Settlements in Unreinforced Areas

An evaluation of settlements in unreinforced areas was completed based on reported estimated and measured settlements found in the literature, as detailed in Table 26 and illustrated in Figure 22. This comparison showed that 6 of the 12 measured settlements were more than estimated, or unconservative. A similar comparison of 124 footing settlements on sands developed by Duncan (2000) resulted in a similar data trend and spread. The 12 data points do trend along the estimated-equals-measured line, which represents the state where estimated settlements equal measured settlements.

Evaluation of the Priebe Method in Reinforced Areas

The evaluation of the Priebe method involved comparing estimated and measured settlements, as detailed in Table 27 and illustrated in Figure 23. From the review of the case history information, the Priebe method was used most. No distinction was made in this study as to which Priebe reference, 1976, 1991, or 1995, was utilized in the case history, as each revision extended the previous procedure. The Priebe method under-predicted the settlements for 6 of the 38 data points, which resulted in field settlements of 110 to 143% of the estimated settlements. The method over-predicted the settlements for 32 of the 38 data points. The settlements were over-estimated up to about 300%. Although most of the data points in Figure 23 are above the estimated-equals-measured line, which indicates a conservative estimation, the writers acknowledge that projects with more settlement than estimated can be considered unsatisfactory and commonly result in the case histories not being published. For example, Raju (1997) mentioned, but did not explicitly detail, two case histories where more settlement was measured than predicted by the Priebe method.

Table 26. Summary of estimated and measured settlements in unreinforced areas

Reference	Project	Type of project	Soil type(s)	Estimating method	Estimated settlement (cm)	Measured settlement (cm)
Greenwood 1970	Bremerhaven Test	Embankment	Soft peat and clay over fine sands	Not Stated	6.7	7.7
Litwinowicz and Smith 1988	Schulz Canal/Pound Creek, Brisbane	Embankment	Soft clays over mudstone	One-Dim. Consol.	110	97
					50[1]	39[1]
					30	25
	Nundah Creek, Western Approach, Brisbane	Embankment	Soft clays over mudstone	One-Dim. Consol.	19[1]	23[1]
					16[1]	25[1]
					22[1]	21[1]
Clemente and Davie 2000	Test Site 1, Area C	2.5-m Square Footing	Sands underlain by soft silts and clays	Elastic	4.4	9.5
	Test Site 2	4-m Square Footing	Silts and clays underlain by loose silts and sands	Elastic	4.5	5.9
	Test Site 3	3.6-m Square Footing	Heterogeneous fill underlain by sands	Elastic	2.0	0.7
					2.6	1.2
Clemente and Parks 2005	Power Plant, England	Storage Tank	Heterogeneous fill underlain by sands	Elastic	3.5	4.7[1]

[1] Settlement shown taken as the average from range of the values reported.

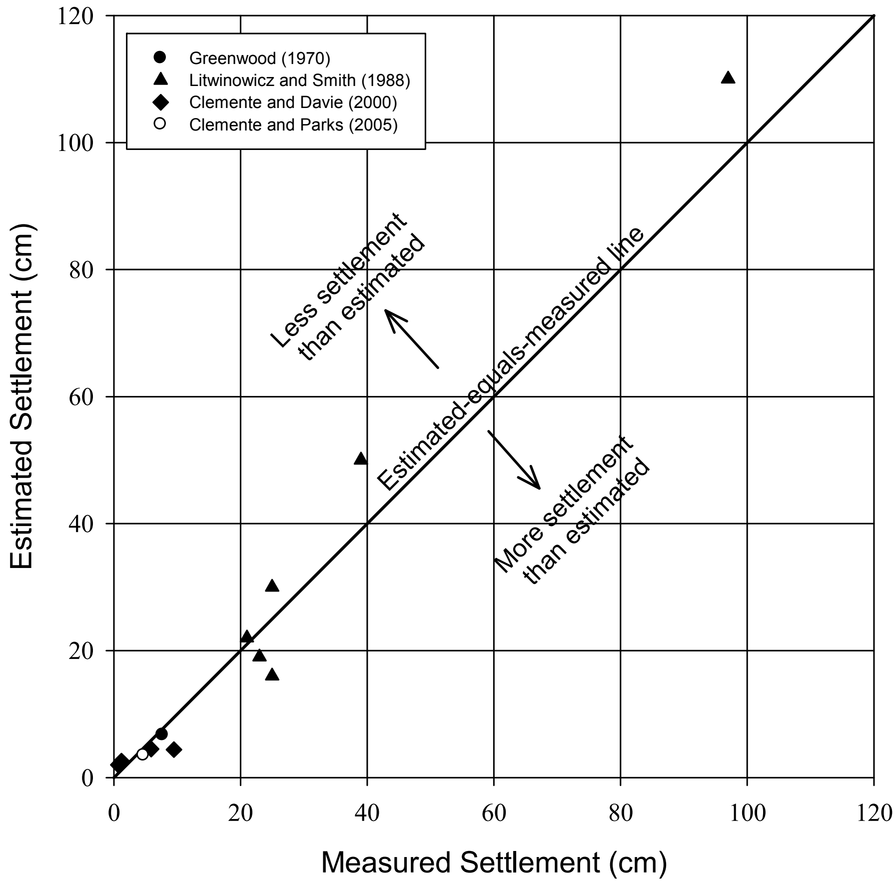

Figure 22. Comparison of estimated and measured settlements in unreinforced areas

A correlation coefficient of variation (COV) of 63% was determined for the estimated-equals-measured line in Figure 23. The correlation COV determination considered the expression "measured settlement equal to estimated settlement," based on the method for computing coefficients of variations of empirical correlations described by Duncan *et al.* (1999) from the work of Ang and Tang (1975). This determination is similar to an evaluation of 54 footings on unreinforced sands with measured settlements greater than 1.3 cm, which resulted in a correlation COV of 67% (Duncan 2000).

Table 27. Summary of Priebe method estimated settlements and measured settlements

Reference	Project	Soil type(s)	Settlement measurement location	Estimated settlement (cm)	Measured settlement (cm)
Greenwood and Kirsch 1984	Silo, Germany	Clays and silts over marl	Foundation	5.3	6.5
	Embankment Fill. Hampton, VA, US	Very soft clay and silt over sand	Center of loaded area	22.2	30
Meade and Allen 1985	US 42, KY, US	Very soft silts and clays	Embankment	20.3	25.4
Kirsch *et al.* 1986	Tank A	Loose/soft very sandy silts	Tank shell	3.0	0.8
	Tank B		Tank shell	3.2	1.0
	Tank C		Tank shell	3.1	1.3
	Tank D		Tank shell	3.1	0.8
	Tank E		Tank shell	2.9	0.5
	Tank F		Tank shell	2.9	0.8
	Tank G		Tank shell	2.9	1.1
	Tank H		Tank shell	2.9	0.8
	Tank I		Tank shell	2.9	1.9
	Tank J		Tank shell	2.4	0.8
	Tank K		Tank shell	2.4	0.5
	Water Tank	Silty sand	Tank shell	6.2	3.2
Clemente and Davie 2000	Test Site 4	Loose sands and silts	Footing Test	1.6	0.2
			Edge of Tank	9.6	2.9[1]
Renton-Rose *et al.* 2000	Plant, Bahrain	Sea dredged sand and gravel underlain by marine sands	Footing Test 1	0.7	0.4
			Footing Test 2	1.7	0.6
			Footing Test 3	0.7	0.2
			Footing Test 4	1.3	0.6
Maduro *et al.* 2004	Multiple Buildings, Puerto Rico	Sand fill, weak silt, sand and peat	Villa Area	12.5	9.1
			S/E Bldg.	38	19
Clemente and Parks 2005	Power Plant, England	Heterogeneous fill underlain by sands	Comb. turbine	4.2[2]	0.9[2]
			Generator	4.4[2]	1.1[2]
			Steam turbine	3.7[2]	1.9[2]
Raman 2006	Railroad, Malaysia	Soft clays and loose sands	Embankment	8.3	4.8
			Embankment	6.4	3.2
			Embankment	8.8	2.9
			Embankment	6.8	2.4
			Embankment	5.6	2.0
Mestat *et al.* 2006; Wehr and Herle 2006	Class A Embankment Settlement Prediction Exercise	Fill, compressible fine grained soils	Centerline	6.4	5
				10.0	12
			Top of slope (at shoulder)	6.4	4.5[2]
				10.0	11[2]

Table 27. (continued)

Reference	Project	Soil type(s)	Settlement measurement location	Estimated settlement (cm)	Measured settlement (cm)
Bouassida *et al.* 2009a; Ellouze *et al.* 2010	Oil Tank, Tunisia	Loose silty sands underlain by marl stone	Edge of tank	2.1	3.0
Bouassida *et al.* 2009b	2-m High Embankment	Soft alluvial clay	Embankment	3.0	1.8
Mohamedzein and Al-Shibani 2011	Embankment	Soft clay underlain by sands	Center of embankment	27.0	24.3

[1] Reported average settlement.
[2] Settlement shown taken as the average from range of the values reported.

In order to evaluate the Priebe method, the 18 data points with measured settlements greater than 1 cm and less than 8 cm were selected. This range represents settlements that would be typical of a serviceability limit state analysis where limiting settlements is a project requirement. A typical settlement limit is 5 cm according to Bond and Harris (2008). In U.S. practice, structural and embankment settlements are typically limited to 2.5 cm and 5 cm, respectively. If settlements exceed 8 cm, stone columns likely are providing stability to the structure or embankment, and settlement determinations do not always control the final configuration. Figure 24 illustrates the data points in this range. A correlation COV of 61% for the estimated-equals-measured line was determined for the data shown in Figure 24.

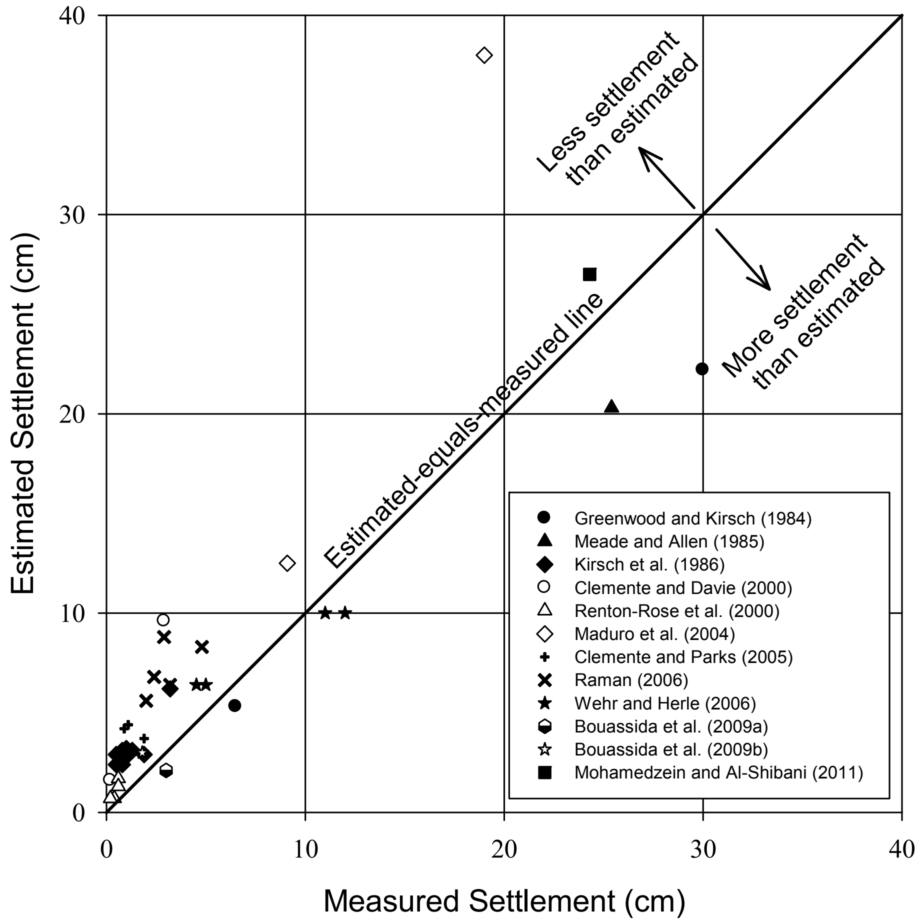

Figure 23. Comparison of estimated and measured settlements in reinforced areas using the Priebe method

A linear regression for the selected data was completed and is shown in Figure 25. With an r^2 of 0.27, the correlation is poor. To enhance the usefulness of the regression, 95% confidence intervals were added to the plot. The 95% confidence intervals shown do not bound 95% of the data, but illustrate the bounds of 95% of the possible regression lines. Based on the location and trend of the regression line and confidence intervals, the Priebe method is shown to provide a conservative design up to settlements of 5 to 6 cm. The regression line trends somewhat parallel the estimated-equals-measured line and indicate that the Priebe method typically over-estimates settlements by 150 to 200%. Elias *et al.* (2006) and McCabe *et al.* (2009) have previously found the Priebe method to provide conservative results.

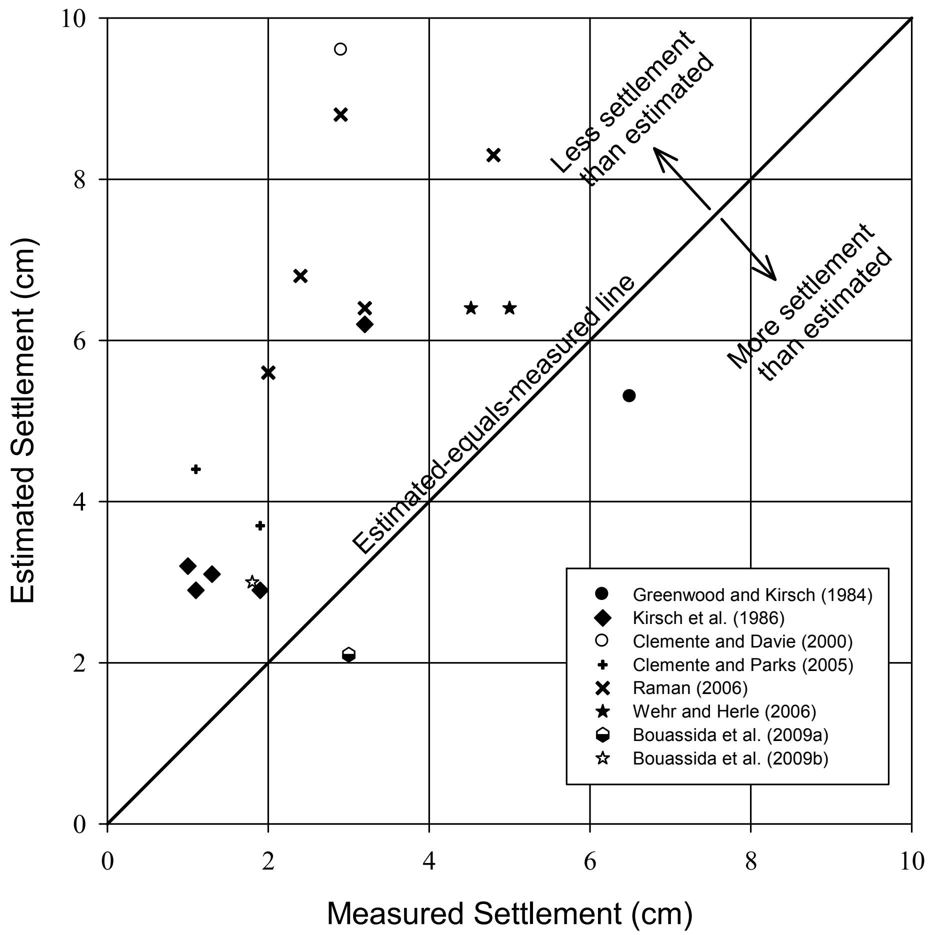

Figure 24. Estimated settlements using the Priebe method for measured settlements greater than 1 cm and less than 8 cm

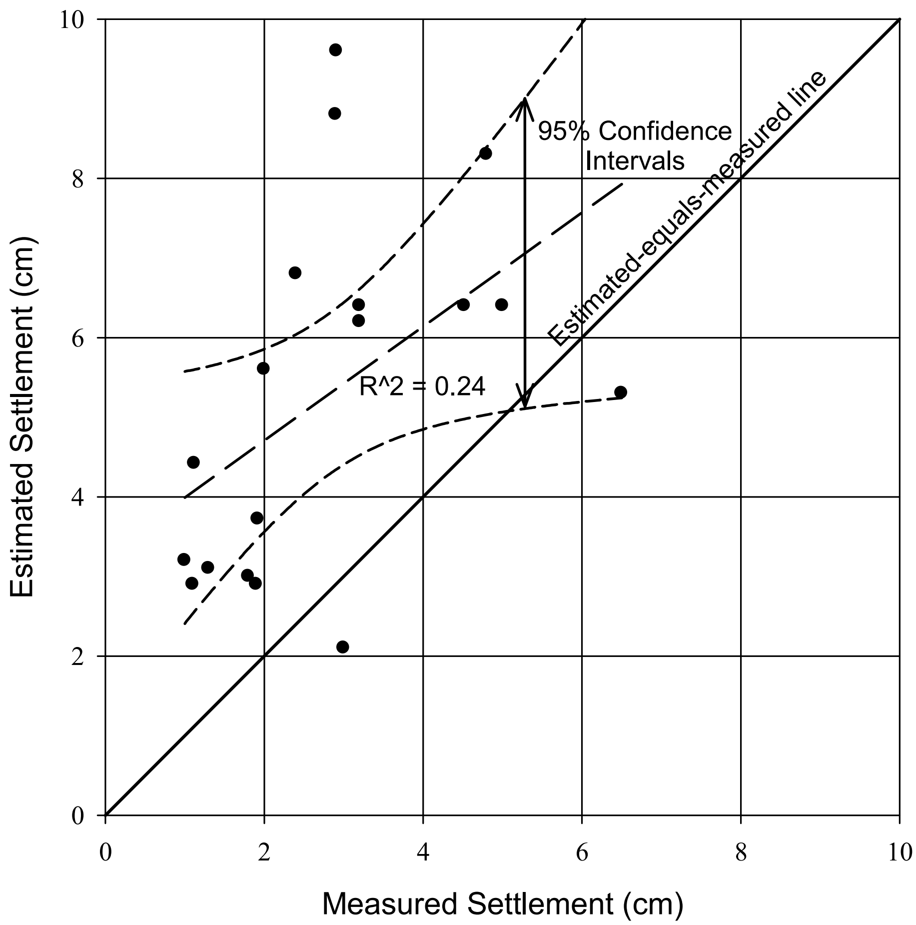

Figure 25. Linear regression and 95% confidence intervals for possible linear regressions using the Priebe method for measured settlements greater than 1 cm and less than 8 cm

Upon initial study of Figure 25, a simple conclusion would be that the Priebe method could be improved. One could hypothesize that an acceptable, or even ideal, estimating method would have a regression line along the estimated-equals-measured settlement line. However, this hypothetical method would result in approximately 50% of the data points in the unconservative range. Should a prudent engineer use that hypothetical method to estimate settlement on a project with strict settlement requirements? To address this question, the concept of reliability is introduced to further evaluate the Priebe method.

Reliability of Settlement Estimates

The prediction of settlements is still difficult. Kirsch and Sondermann (2003) estimated reinforced ground settlements below an embankment using the Priebe method to be 192 cm compared with 240 cm estimated with a finite element model and concluded that the match “appears satisfactory.” Example calculations presented by Priebe (1995) estimated a final settlement of 38 cm where Greenwood (1991) measured settlements of 40 to 41 cm. Thus, even the example calculation shown by Priebe (1995) under-predicted the known settlement by 5 to 8%.

The Priebe method data shown in Figures 2 and 3 resulted in correlation COVs of 63 and 61%, respectively. Considering the reliability approach presented by Duncan (2000) with a 60% COV for the Priebe method, there is a 10% probability that the settlement may be larger than 175% of the estimated settlement. With this 10% possibility of much larger settlements than estimated, a prudent designer would be compelled to consider the following questions:

- How much variation is inherent in the estimating method?
- How much variation is inherent to in situ properties?
- Should estimated settlements be presented as a single number or a range?
- What are the consequences if the settlements are under-predicted?
- What percent probability for exceeding a settlement threshold is the designer or owner willing to accept?

The concept of reliability can assist in answering these questions. Although the Duncan (2000) approach elicited much discussion (see discussion to Duncan 2000), Christian and Baecher (2001) described Duncan's approach in the discussion as "a straightforward exposition of reliability methods without mystification." Prior to discussing the Priebe method in terms of reliability, observed variations from reported projects can provide a reference for comparing the expected variability of the estimating process to the variability of site conditions.

Field Variability from Case Histories

The results of monitoring an unreinforced water storage tank described by Clemente and Parks (2005) yielded settlements ranging from 2.5 cm to 6.8 cm across the 17.5-m diameter tank. Settlements were estimated to be 3.5 cm. The measured settlements were 71 to 193% of the estimated settlement for the unreinforced water tank. This example is provided to illustrate (1) that much larger settlements than predicted are possible across an individual project site, and (2) that the probability of settlements in excess of 175% of the estimated settlements are real as modeled using reliability by Duncan (2000).

The field test for stone column reinforced ground in Hampton, Virginia, included a loaded area 6.1 m by 6.1 m, as presented by Goughnour and Bayuk (1979b). Settlements were monitored below the center and at the four corners of the loaded area. The settlements at the four corners after 130 days were 8.1 cm, 9.7 cm, 12.5 cm, and 13.2 cm. From these four readings, the site COV for settlement was 22%. This value compares well with an unreinforced case history by Wu *et al.* (2011), which found a site COV for settlement of 21%. The site COV is due to the change in soil parameters and profiles across the site. Note that if the rate of consolidation is included in the evaluation, a much larger site COV will result due to the variable drainage conditions (Alonso and Jimenez, 2011).

Reliability of the Priebe Method

With a correlation COV of 60% for the Priebe method considering the estimated-equals-measured line, Figure 26 illustrates how probabilities of exceedance of 1, 5 and 10% compare with selected data from the case histories. These probabilities result in estimated settlements 175 to 300% of the values along the estimated-equals-measured line. Figure 26 represents the highest likely COV from the Priebe method.

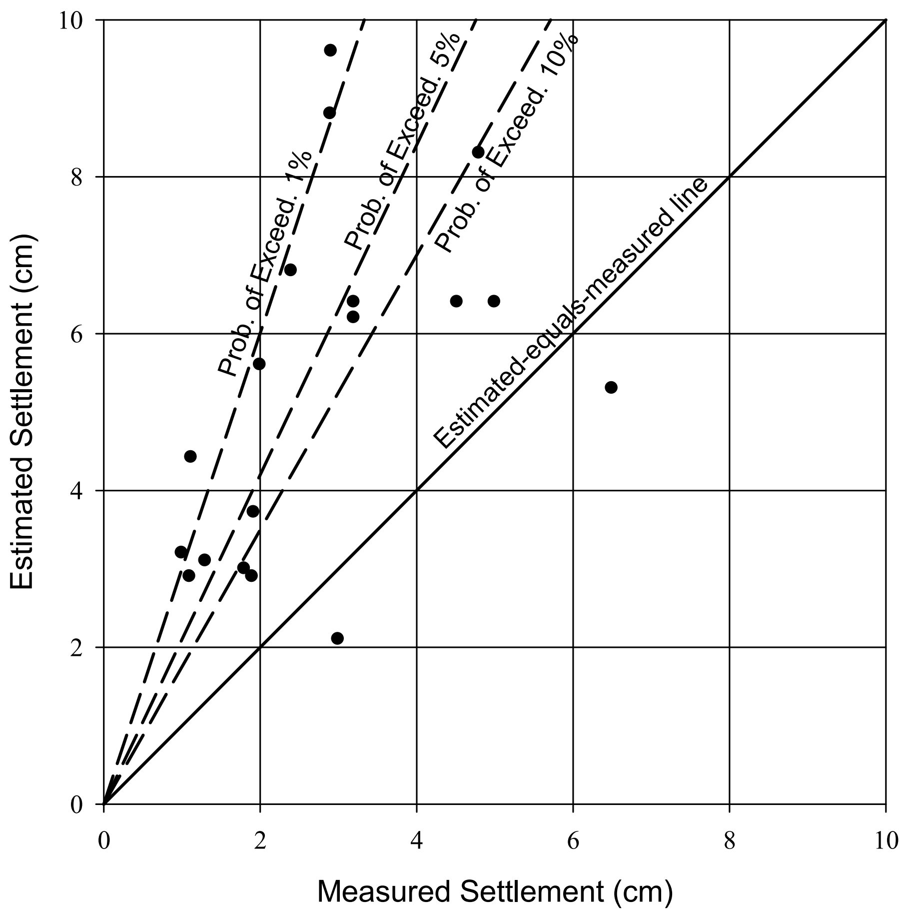

Figure 26. Reliability of the Priebe method estimates considering a COV of 60% with regard to the variation of the data to the estimated-equals-measured line

Inherent site variability results in a site COV of 20 to 25%, regardless of the analysis method. This site COV represents the lowest variation that could be reasonably assumed in a settlement evaluation. Considering a COV of 25%, Figure 27 graphically illustrates how probabilities of exceedance of 1, 5 and 10% compare with selected data from the case histories. Consideration of these probabilities results in estimated settlements 135 to 175% of the values along the estimated-equals-measured line.

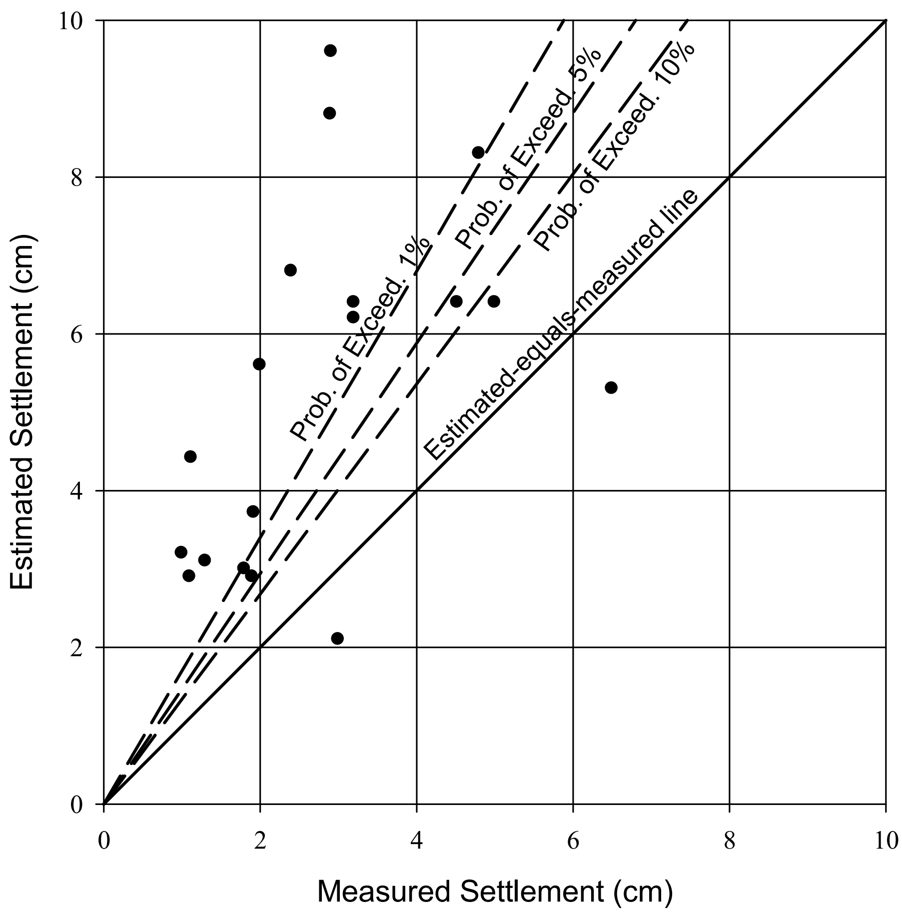

Figure 27. Reliability of the Priebe method estimates considering a COV of 25% with regard to site soil conditions

Through iteration of lines similar to the probability lines of Figures 26 and 27 and determining the correlation COV of each possible line using the Ang and Tang (1975) approach, the best fit line is shown in Figure 28 and resulted in a correlation COV of 48%.

The concept of provides an assessment of the conservativeness of the Priebe method. Considering a correlation COV of 50% for the best fit line, there is an 89% probability that settlements will be smaller than those estimated with the Priebe method. Or stated differently, the Priebe method tends to over-estimate measured settlements by approximately 160%.

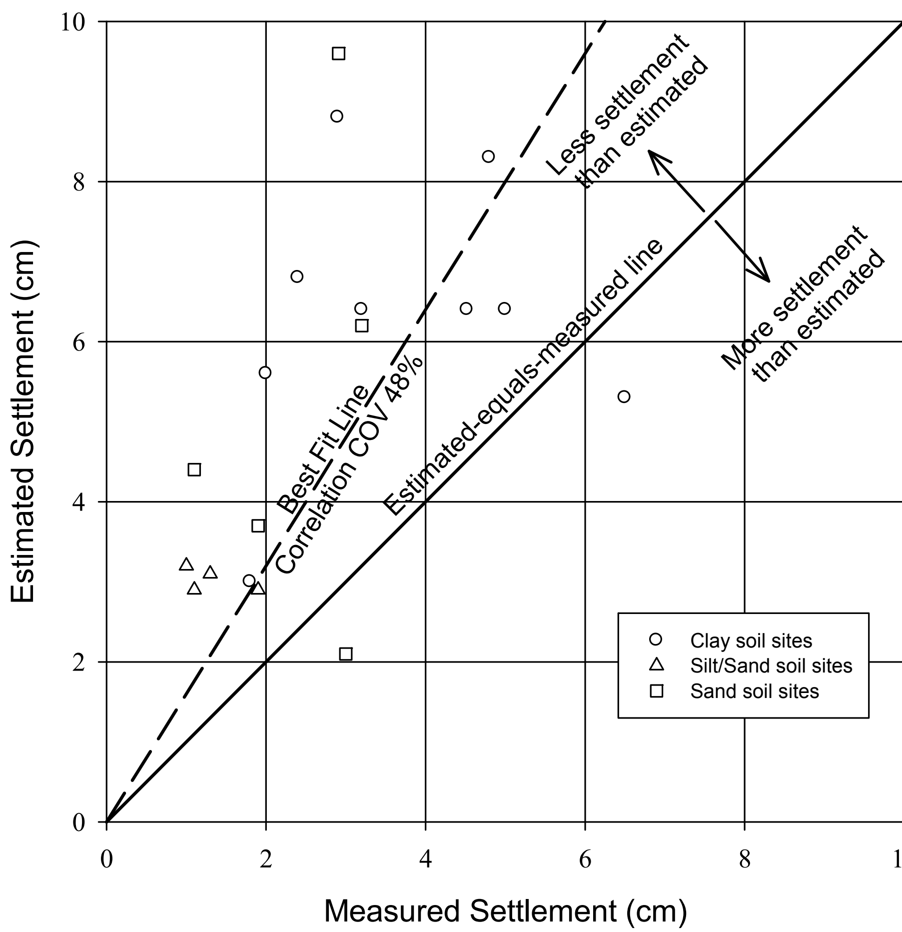

Figure 28. Priebe method best fit line and data sorted by site soil conditions

The evaluation included case histories in which clay, silt/sand, and sand soils were reinforced with stone columns. Figure 28 also illustrates the performance of stone columns in the different soil types. Clear conclusions regarding the Priebe method and its applicability to different soil types could not be developed from Figure 28.

Conclusions

Stone column case histories provide highly variable information regarding site conditions, soil parameters, design considerations, construction process, and settlement monitoring. This study confirms the work of previous authors that there is a lack of detailed, research-oriented case histories that fully document the design, construction, and performance of stone column reinforced ground (Barksdale and Bachus 1983; McCabe *et al.* 2009).

The Priebe method is the most common method used in practice for estimating settlements of stone column reinforced ground. Even though the Priebe method does not capture all the parameters that affect the performance of stone column reinforced ground, the method is preferred due to its simplicity. Since no safety factors or margin of error are currently considered in settlement analyses, reliability provided a framework to evaluate the Priebe method. Considering data with a maximum measured settlement of 8 cm, there is an 89% probability that settlement estimated with the Priebe method will exceed measured values. As shown in the case histories and the reliability study, the Priebe method is not always conservative, and settlements may exceed those estimated. As more complicated models and methods continue to be developed, the geotechnical community should consider if less conservative methods than the Priebe method should be used to estimate settlements of stone column reinforced ground.

A philosophical statement from Terzaghi (1936) over 75 years ago may be the best reminder to practicing engineers when contemplating settlements: "Whoever expects from soil mechanics a set of simple, hard and fast rules for settlement computation will be deeply disappointed…. The nature of the problem strictly precludes such rules."

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CHAPTER 6. STONE COLUMNS: LESSONS LEARNED, SETTLEMENTS, AND FUTURE PROJECT CONSIDERATIONS

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S. Caleb Douglas[17 18], P.E. and Vernon R. Schaefer[19 20], P.E., M. ASCE

Abstract

Stone columns have been successfully used for transportation projects across the United States to treat clays, clayey sands, and silty sands for over three decades. However, stone column–specific knowledge is generally accessible to only a select group of stone column experts and specialty contractors. Data mining identified numerous case histories that allowed both lessons learned to be compiled from projects that encountered unsatisfactory performance and current settlement estimating methods to be evaluated. The unsatisfactory performance revealed inadequacies in three broad aspects of every project: site investigation,

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[17] Ph.D. candidate, Civil Engineering, Iowa State University, Ames, IA 50010-3232. E-mail: calebd@iastate.edu
[18] Primary researcher and author
[19] Professor of Civil Engineering, Iowa State University, Ames, IA 50010-3232. E-mail: vern@iastate.edu
[20] Secondary author, author for correspondence

design, and construction monitoring. Current methods of estimating settlements were evaluated with a case history and indicated that the Priebe method was preferred for day-to-day designs. Numerical modeling confirmed that the change in stress resulting from the surface load in the layers underlying the stone column reinforced ground can be approximated using a traditional elastic, Boussinesq-type stress distribution.

Introduction

A considerable amount of future highway construction, reconstruction, and widening work will be required in the United States (U.S.) to maintain and expand the transportation network (Elias et al. 2006). Weak soils are commonly encountered on all types of transportation projects. Over the past 30 years, stone columns have been successfully used in transportation projects across the U.S. to treat clays, clayey sands, and silty sands. The advantages of stone columns include increasing bearing capacity, increasing global stability, and decreasing settlements. The development of stone columns, including both the vibro-replacement and vibro-displacement techniques, has been well described by Barksdale and Bachus (1983), Charles and Watts (2002), Elias et al. (2006), and Kirsch and Kirsch (2010). McCabe et al. (2009) provide a recent description of construction methods for stone columns.

Ground strengthened by arrays of columns behaves in complex ways under widespread vertical loads (Greenwood 1991). An initial hurdle for practicing engineers unfamiliar with stone columns is to gain access to stone column–specific knowledge usually confined to a select group of stone column experts and specialty contractors. Additionally, current design methods do not fully capture the behavior of stone column reinforced ground. Even with three decades of use in the U.S., stone column design remains semi-empirical. A number of case histories with unsatisfactory performance allowed problematic conditions for stone columns to be identified. Application, design, and construction considerations for future projects were developed from these problematic conditions and represent a summary of lessons learned from previous projects.

The literature over the last two decades is clear on the inadequacies of present settlement prediction methods for ground reinforced by with stone columns (Allen et al. 1991; Clemente and Davie 2000; Abdrabbo and Mahmoud 2002; Raman 2006; Mestat et al. 2006; McCabe et al. 2009). Over 15 design methods have been reported to estimate the settlement of soil reinforced this way (Greenwood 1970; Hughes and Withers 1974; Priebe 1976, 1991, 1995; Goughnour and Bayuk 1979; Balaam and Booker 1981, 1985; Balaam and Poulos 1983; Barksdale and Bachus 1983; Van Impe and De Beer 1983; Chow 1996; Alamgir et al. 1996; Poorooshasb and Meyerhof 1997; Pulko and Majes 2005; Ambily and Gandhi 2007; Borges et al. 2009). Current Federal Highway Administration (FHWA) guidance is to estimate a range of possible settlements using the Priebe (1995) method to evaluate the upper bound effectiveness of stone and the Equilibrium method to evaluate the lower bound effectiveness (Elias et al. 2006). The current FHWA recommendations for estimating settlements were evaluated with a well-documented case history, which also contributed to the lessons learned. The stress distribution below the stone column reinforced zone due to the widened embankment was studied through finite element modeling. Specific guidance for estimating settlements resulted from the evaluation and modeling of the case history.

Defining Stone Column Performance

The performance of stone column reinforced ground can be introduced using the concepts of ultimate limit state and serviceability limit state. Satisfactory performance of stone column reinforced ground requires consideration of both limit states in design. Stone column projects require design against a complete or partial failure and are commonly addressed in global stability and bearing capacity analyses. Ultimate limit state failures typically involve a loss of static equilibrium. These complete and partial failures are addressed with the concept of the ultimate limit state. Gue and Tan (2003) reviewed 55 geotechnical failures and found that about one-third of geotechnical failures were the result of inadequate design with reference to the ultimate limit state. Equally important is the serviceability limit state. The serviceability limit state requires that under the project loading conditions, the resulting deflections do not exceed a threshold limit. The threshold limit can be for the purpose of minimizing pavement distresses or some other requirement as specified by the owner or designer. Gue and Tan (2003) found that about two-thirds of geotechnical failures were the result of inadequately designing the project with respect to the serviceability limit state.

Review of Prior Issues and Failures

Case histories that describe unsatisfactory performance provide valuable information that designers should consider for future projects. Unsatisfactory performance has resulted from problematic conditions that include site conditions, design, construction, or quality control/quality assurance (QC/QA). Table 28 provides a summary of problematic conditions identified from the review of case histories. Five references from Table 28 are briefly summarized in the following sections to emphasize that proper site characterization and engineering judgment must be incorporated into the design and construction process.

Summary of Two Projects in the United Kingdom

Charles and Watts (2002) summarized two projects in the United Kingdom. First, a housing development was constructed on a site with existing fill. The 10-m (33-ft) thick fill was composed of intermixed lumps of stiff clay and weathered mudstone. Stone columns were installed below the house foundations, which consisted of stiff rafts. The houses experienced tilts on the order of 30 cm (12 in) over a distance of 9 m (30 ft). Forensic study concluded the stone columns provided pathways for water to infiltrate the fill soils, which resulted in collapse compression of the fill. Charles and Watts (2002) warned of the mitigation costs involved to repair structures if sitea are not adequately studied prior to design and construction of stone columns.

Charles and Watts (2002) also described a service station built on fill placed on a natural slope. A station was initially constructed in 1963, but following the placement of additional fill substantial ground movements occurred. An investigation concluded that the fill had slipped at the contact with the natural slope. The old service station was razed in 1984, and a new service station constructed around 1987 on stone columns to reinforce the existing fill. The new structure then experienced horizontal and vertical movements. A study revealed that the site was located on a pre-existing landslide. Several of the ground improvement contractors invited to bid on the project stated that stone columns were not appropriate and that a stability analysis was required (Charles and Watts 2002).

Table 28. Problematic conditions and future project considerations

Problematic condition	Reference (s)	Future project considerations
Lack of adequate geotechnical investigation	Meade and Allen 1985; Slocombe 2001; Charles and Watts 2002	A detailed geotechnical investigation is required for stone column projects. No areas of serious doubt should exist within the area to treat.
Stiffer soils encountered during construction which slowed installation	Meade and Allen 1985	
Sensitive soils	McKenna et al. 1975; Wijeyakulasuriya et al. 1999; Gue and Tan 2003; Oh et al. 2007a, 2007b	Stone columns should be used with caution on projects with sensitive clays as clays will be weakened during installation.
Thick peat deposits	Slocombe 2001	Peat layers have to be accommodated and considered in design and construction.
Very soft soils with shear strengths as low as 5 to 6 kPa (100 to 125 psf)	Raju et al. 2004; Serridge and Synac 2007	This is a very advanced application of the technique and requires experienced designers and contractors.
Fill heterogeneity	Clemente and Davie 2000; Slocombe 2001	The variability of the fill can result in installation issues and a very wide range of support conditions can result.
Collapsible soils or fills	Charles and Watts 2002; Slocombe 2001	Stone columns have the potential to supply water to the soils which can result in collapse.
Lack of global stability considerations	Charles and Watts 2002; Gue and Tan 2003	Designers must consider all possible scenarios which affect a project site.
Lack of expected improvement at edge of reinforced area	Cooper and Rose 1999	A reduced efficiency of stone columns along the edges of a widely-reinforced area are possible.
Lack of acknowledging contractor comments during bidding	Charles and Watts 2002	Input from experienced contractors should be considered by the designers.
Weakening of in situ soils during installation	White et al. 2002; Chen and Bailey 2004; Kirsch 2006, 2009	Stone columns installed into stiff to hard soils can result in a weaker soil structure.
Lack of construction supervision by engineer of record	Gue and Tan 2003	QC/QA is essential to satisfactory performance of stone columns. The geotechnical engineer of record should be included in QC/QA activities.
Poor construction quality	Bell 2004	
Loading rate did not allow dissipation of excess pore water pressures	Greenwood 1991; Chummar 2000	Analysis must consider reduction in strength of in situ soils upon loading. Construction should be overseen by experienced geotechnical engineer using data from piezometers and settlement plates.
Quick, small-scale load test	Greenwood 1991; Chummar 2000	The scale of any load test should be representative of project conditions. Small scale tests are appropriate only if they simulate prototype loading in every respect.
Stone columns became fouled at surface and did not allow drainage	Chummar 2000	The stone columns should be directly connected to the drainage blanket and construction should not allow the tops of the stone columns to become fouled.
Ground disturbance adjacent to stone columns	Venmans 1998	Projects should include repair or replacement plans for items such as road signs which can be damaged by heaving ground.
Slope movements during construction	Rosidi et al. 2008	Identification and monitoring of adjacent slopes which are susceptible to movements induced by the installation vibrations.

Failures of Embankments A and B

Gue and Tan (2003) describe the failures of two embankments which were constructed over stone columns. The two embankments were identified as Embankments A and B in the reference. The geographic locations of the embankments were not provided.

The soils at Embankment A consisted of very soft silty and sandy clays underlain by a thin layer of very loose clayey sand. Medium stiff to stiff silty clay and clayey silts were located below the clayey sand. The soils from the ground surface to a depth of 16 m (52 ft) had a minimum sensitivity of 2 and a maximum of 26, with the majority of data indicating sensitivities of 5 to 12. Vacuum preloading with prefabricated vertical drains was used initially at Embankment A (Gue and Tan 2003). The embankment experienced a global failure during construction. A remediation treatment for the failed embankment using stone columns was designed and constructed. The stone column reinforced embankment then also failed globally during reconstruction when the embankment reached 3.2 m (10 ft) of the 5.5 m (18 ft) planned fill height. Upon review of the design process after failure, only the Priebe method was used to estimate settlements with no consideration for the stability of the embankment (Gue and Tan 2003).

Embankment B was approximately 2 km (1.2 miles) from Embankment A. Embankment B was initially treated with prefabricated vertical drains and surcharging (Gue and Tan 2003). The natural soils consisted of an organic soil with a thickness of about 4 m (13 ft) underlain by 10 m (33 ft) of very soft to soft silty clay followed by stiff to very stiff silty clay. The planned grade required an embankment height of 2.4 m (8 ft). A slip failure occurred through the soil treated with prefabricated vertical drains during placement of the surcharge. The contractor then reinforced the soils with stone columns so that the embankment could be reconstructed. However, the embankment supported by the reinforced ground also experienced a slip failure when the fill reached the full surcharge height of 3.9 m (13 ft). Again, design considered only settlements with no consideration for ultimate limit states (Gue and Tan 2003).

Coombabah Creek Test Embankment, Australia

A trial embankment was completed at the crossing of Coombabah Creek in southeast Queensland, Australia (Wijeyakulasuriya et al. 1999; Oh et al. 2007a, 2007b). The trial embankment was constructed in a swamp with up to 13 m (43 ft) of soft clay and lacked a weathered surficial crust. The soft, estuarine silty clays typically had undrained shear strengths around 10 to 15 kPa (200 to 300 psf). Field shear vane tests yielded sensitivities between 5 and 13 with an average of 6. The trial embankment consisted of two 12-m (40-ft) long sections reinforced with 1-m (3.3-ft) diameter stone columns at 2-m (6.6-ft) and 3-m (9.8-ft) square spacings, and a third section that was unreinforced ground. The stone columns were installed using the vibro-replacement process. The trial embankment was 2 m (6.6 ft) high with a top width of 12 m (40 ft). All three test sections resulted in similar time-versus-settlement curves. Poor performance was attributed to weakening of the sensitive clays during stone column installation (Wijeyakulasuriya et al. 1999; Oh et al. 2007a, 2007b).

"Routine" Foundation Project, United Kingdom

Load tests to verify the performance of a vibrated stone columns did not pass the load and deformation requirements (Bell 2004). After the load tests failed, an investigation was initiated to expose, excavate, and observe several columns along their axes. This investigation indicated that many columns were poorly constructed. Some columns were not continuous with depth. Some columns had smaller diameters than the design diameter. Other columns had a top diameter as designed, but the diameter continuously reduced with depth, resulting in a smaller bottom diameter than designed. The production columns were subsequently constructed with the same equipment that was used to construct the unsatisfactory columns, but a higher quality standard was implemented to construct the columns as a result of the investigation (Bell 2004).

Highway A2, Netherlands

With stone columns having limited usage in the Netherlands in the 1990s, an extensive field test program was established to evaluate the use of stone columns along a highway widening project (Venmans 1998). The soil profiles consisted of organic clay to 0.5 m (1.6 ft) underlain by peat and soft silty clay to 5 m (16 ft). The clay was very soft, with undrained shear strengths on the order of 15 to 20 kPa (300 to 400 psf). Installation and construction problems were encountered during construction of the stone columns. Installation of the stone columns was cumbersome in the very soft clays. Columns with lengths greater than 4 m (13 ft) could not be constructed. Fracturing of the ground resulted from high water pressures used while advancing the vibroflot. The traditional local method of widening included preloading with prefabricated vertical drains, and the damage to the existing pavement was greater than with the traditional method. All road-side sign foundations were damaged during stone column installation and had to be replaced (Venmans 1998).

Applicability and Serviceability Considerations

The poor performance revealed inadequacies in three broad areas, which are site investigation, design, and construction monitoring. The benefit of Table 28 is the compilation of problematic conditions that will alert a designer to a potential misapplication of stone columns. After a designer identifies stone columns as a potential solution for a project, design must consider both ultimate and serviceability limit states. To assist designers in applying stone columns to serviceability requirements, the typical range for the reduction of settlements is evaluated using a case history. The estimated and measured settlements for the US Highway 42 embankment widening project as reported by Meade and Allen (1985) are representative of the expected improvement from the use of stone columns. The unreinforced settlements were estimated to be 56 cm (22 in), and the reinforced settlements were measured to range from 18 to 20 cm (7 to 8 in), which is approximately 32 to 36% of the estimated unreinforced settlements. Stone columns typically reduce settlements to 30 to 50% of the estimated unreinforced settlement (Elias et al. 2006). The amount of improvement has been shown to be primarily dependent on the stone column diameter and spacing, which relate to the amount of area replaced with stone.

US Highway 42 Case History

A well-documented embankment widening case history for US Highway 42 in Gallatin County, Kentucky, was completed by Meade and Allen (1985). The project required a new embankment to be constructed adjacent to an existing embankment to provide for a new bridge approach. The existing embankment was supported on an unreinforced foundation. The widening portion was located in a backwater area of an adjacent river. The soils were very soft silty clays that extended from the ground surface to a depth of about 7.6 m (25 ft). A cross-section of the project with soil conditions is provided in Figure 29. The project designers used stone columns to increase the global stability and decrease the settlement of the widened portion of the embankment. Both vertical and horizontal movements were measured during construction of the embankment. The case history provides the data to evaluate current FHWA settlement methods.

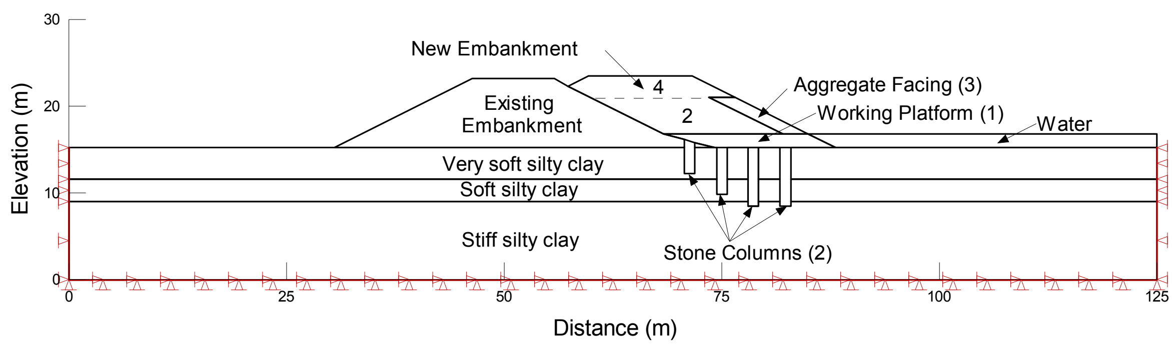

Figure 29. Cross-section of project and boundary conditions for finite element model

The stone columns were designed to have a 1.1-m (3.5-ft) diameter placed in a triangular pattern at a spacing of 2.1 m (7 ft). The diameter of stone columns as constructed was 1.2 m (4 ft). Vertical deformations were measured using settlement plates placed on top of the working platform after installation of the stone columns. Horizontal deformations were measured using a slope inclinometer placed near the toe of the slope.

Methods of Estimating Settlement

The recommended design procedure for estimating settlement presented by Elias et al. (2006) in the FHWA *Ground Improvement Methods* manual is to use the Priebe method to evaluate the lower bound settlement and the Equilibrium method to evaluate the upper bound settlement.

Equilibrium Method

The Equilibrium method is a simple procedure for estimating the settlement of stone column reinforced ground. In using this approach, the stress concentration ratio must be estimated using either experience or the results of field stress measurements, such as those obtained from full-scale embankments. The stress concentration ratio (SCR) is the stress on the stone column divided by the stress on the soil. Lower estimates of stress concentration factors result in more conservative (larger) settlement predictions. The equilibrium method is detailed in Barksdale and Bachus (1983) and Elias et al. (2006) and generally includes the following steps:

1. Estimate a value to be used as the SCR. Barksdale and Bachus (1983) suggested a SCR of 4 to 5. Elias et al. (2006) suggested a SCR of 2.5 for preliminary design and that a typical range is 2 to 4.
2. Calculate the area replacement ratio using the stone column diameter and spacing.
3. Determine the resulting final vertical stress on the soil between the columns using the SCR and the area replacement ratio.
4. Use the stress on the soil and conventional one-dimensional consolidation theory or elastic theory to estimate the settlement of soil.

Priebe Method

Priebe (1995) provided design procedures and design charts for various aspects of stone column design, including settlement reduction, bearing capacity, shear values of improved ground, settlement of footings, and liquefaction. Priebe (1995) stated, "The design method refers to the improving effect of stone column in a soil which is otherwise unaltered in comparison to the initial state." The assumptions and procedures associated with analyzing the reduction in settlements are well documented in Priebe (1995), and the reader is referred to that paper for a description of the estimating procedure. The Priebe method results in an improvement factor based on the area replacement ratio and strength of the column material. The Priebe method generally includes the following steps:

1. Estimate settlements of the unreinforced soil using either consolidation or elastic theory. The constrained modulus is typically used in the elastic approach.
2. Calculate the area replacement ratio using the stone column diameter and spacing.
3. Determine the improvement factor. An improvement factor can be determined for each soil layer considered in the settlement analysis.
4. The estimated reinforced settlement is calculated using the determined improvement factors for the reinforced zone and traditional settlement calculations below the reinforced zone.

Estimated and Measured Settlements

The Kentucky Department of Highways, Division of Materials, estimated a settlement of 40.5 cm (16 in) using the Equilibrium method and a stress concentration ratio of 3. GKN Keller, the stone column contractor, estimated a settlement of 20 cm (8 in) using the Priebe method. The maximum measured settlements at the top of the working platform post installation of the stone columns were in the range of 18 to 20 cm (7 to 8 in). Meade and Allen (1985) noted that construction of the working platform and stone columns resulted in settlement of the underlying soils, and this settlement was not captured in the settlement record.

Although not explicitly discussed in Priebe (1995), matching of the stresses in the example calculations provided in Priebe (1995) indicate that a Boussinesq-type analysis was utilized to estimate the stresses in the unreinforced soils. Neither the Equilibrium method nor the Priebe method directly addresses the influence of the stone columns on the stress distribution within the reinforced soils and the underlying unreinforced soils. The US Highway 42 case history with measured vertical and horizontal settlements allowed finite element modeling to evaluate the influence of the stone columns on the stress distribution.

Finite Element Modeling

The stress distribution below stone column reinforced ground has not been well documented in the literature. Stress distribution approximations were presented for similar aggregate column systems by Aboshi et al. (1979), Bowles (1982), Fox and Cowell (1998), and Sehn and Blackburn (2008). To address this knowledge gap, a finite element model (FEM) was developed using the SIGMA/W program (GeoStudio, Version 7.16) to provide guidance regarding the stress distribution using the data from the US Highway 42 project. The material properties utilized in the model are provided in Table 29 and correspond to the labels in Figure 29. The elastic-plastic soil properties were initially established utilizing the soils information from the project history. A plane strain analysis was completed, and the three-dimensional problem was converted to plane strain using the scheme described by Tan and Oo (2008). The stone columns were modeled in two dimensions as trench widths of 0.66 m (2.2 ft). Both triangular and quadrilateral elements were used with a mesh size of 0.25 m (0.82 ft). The boundary conditions shown in Figure 29 represent the bottom to be fixed both vertically and horizontally and the sides to be fixed only horizontally.

A drained analysis was completed with four stages of loading. The numbers shown in Figure 29 indicate the stage where the zone was incorporated into the model. Prior to any loading, the initial in situ stresses were calculated based on the existing conditions. Each successive loading was applied as the body weight of the material being added in that stage. Each stage utilized the effective stresses from the previous stage. Stage 1 loading consisted of placing the working platform. Stage 2 consisted of placing the stone columns into the model and applying the "core" of the new embankment. Stage 3 consisted of placing the granular facing on the slope. Stage 4 placed the upper portion of the fill to final grade.

Table 29. Material properties utilized in finite element model

Layer description	Model development	Moist unit weight (kN/m^3)	Modulus of elasticity, E (kPa)	Cohesion, c' (kPa)	Angle of internal friction, ϕ' (degrees)	Poisson's ratio, ν	Dilation angle, ψ (degrees)
Existing embankment	Initial estimate	20	15,000	15	30	0.33	0
	Final model	20	15,000	15	30	0.33	0
Working platform and facing	Initial estimate	21	30,000	0	38	0.33	0
	Final model	21	40,000	0	36	0.33	0
New embankment	Initial estimate	20	15,000	15	30	0.33	0
	Final model	20	15,000	15	30	0.33	0
Very soft silty clay	Initial estimate	18	1,000	0	25	0.2	0
	Final model	18	3,400	0	20	0.33	0
Soft silty clay	Initial estimate	19	10,000	0	30	0.25	0
	Final model	19	3,400	0	30	0.33	0
Stiff silty clay	Initial estimate	20	30,000	0	32	0.33	0
	Final model	20	60,000	0	32	0.33	0
Stone columns	Initial estimate	21	30,000	0	42	0.33	12
	Final model	21	40,000	0	42	0.33	12

After the model and staged analyses were functioning, the soil properties were adjusted to match both the measured lateral and vertical deformations. The vertical deformations, or settlements, were measured using settlement plates placed along the top of the working platform. The lateral deformations were measured in the field using a slope inclinometer near the toe of the slope as shown in Figure 30. The goal was to match maximum values and trends in the field data using the numerical model shown in part (b) of Figure 30. The soil properties of the final model are provided below the initial estimates in Table 29.

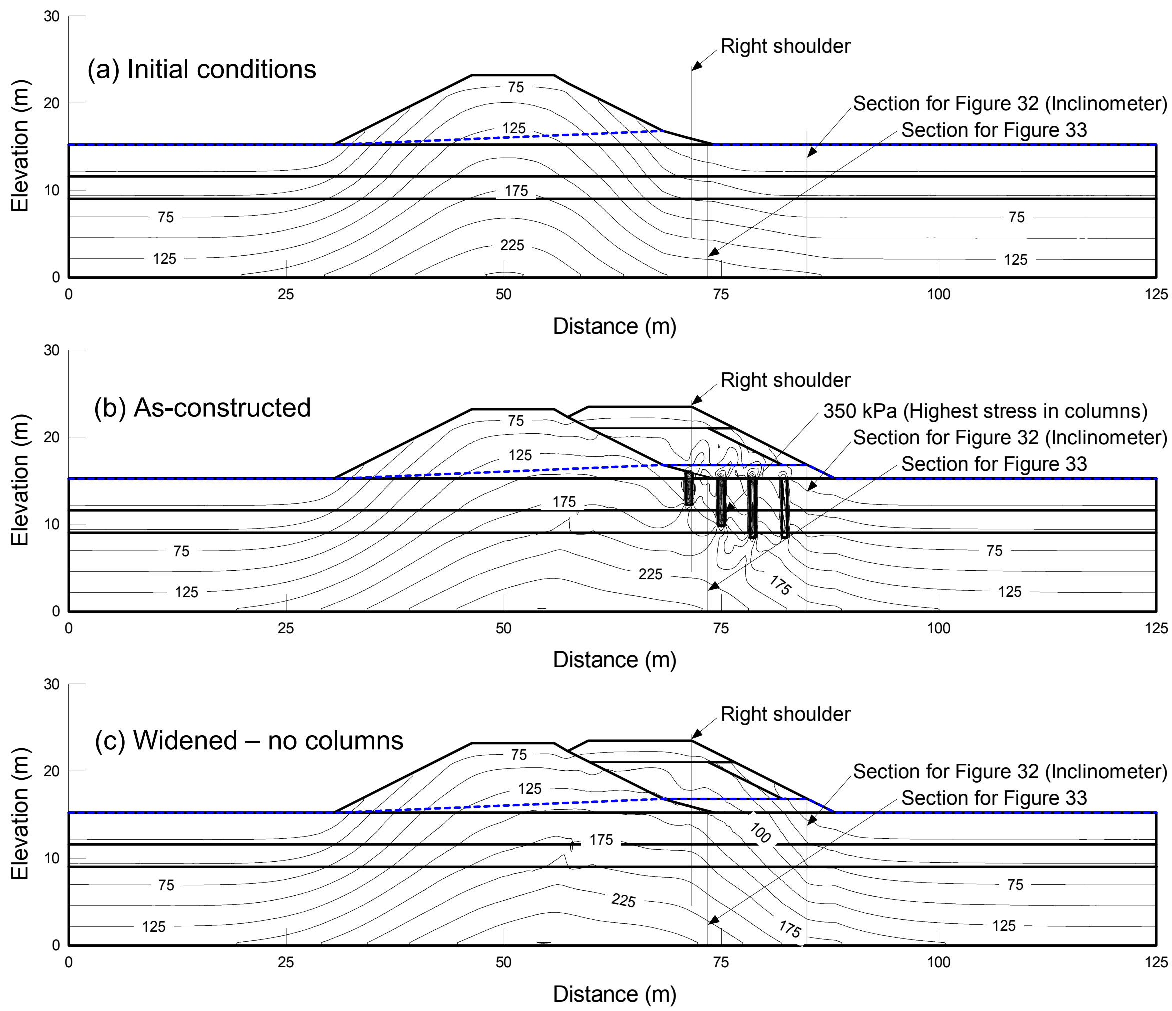

Figure 30. Effective stress contours (kPa) for (a) initial conditions, (b) as-constructed widened section with stone columns, and (c) the widened section without stone columns

Figure 31 compares the measured and modeled settlements. The model does capture the trend of the settlements, but the magnitudes of the settlements approaching the maximum value are not well simulated. The measured and modeled horizontal deformations are shown in Figure 32. The model approximates the lateral deformation in the natural soils, but the model could not fully predict the deformations within the working platform. After the as-constructed model had been calibrated, the stone column elements were removed, and the initial and unreinforced conditions were examined as illustrated in parts (a) and (c) of Figure 30, respectively. The results of these analyses are also included in Figures 31 and 32. When the model was switched to the case without stone columns, more settlement would have been expected than predicted in Figure 31. This illustrates the difficulty in obtaining realistic soil parameters to utilize in a numerical model for estimating both reinforced and unreinforced settlements. However, the intent of this analysis was to evaluate stress distribution, and numerical modeling does provide insight on that subject (Barksdale and Bachus 1983; Ashmawy et al. 2000).

Figure 30 illustrates the distribution of effective stresses within the soil for the initial conditions, the as-constructed embankment with stone columns, and the widened embankment without stone columns. Of particular interest is the change in stress in the zone below the stone columns, which typically must be considered in the settlement analysis. A plot of effective stresses for each of the three conditions was determined with depth within the natural soils, as shown in Figure 33. The section that was selected is shown in Figure 30 and corresponds to the point under the working platform that was modeled to have the highest settlement. Comparison of the analyses for the embankment with stone columns to the embankment without stone columns indicates that the vertical effective stresses in the in situ soil in the reinforced zone have a maximum reduction of approximately 20% with the inclusion of stone columns, and the soil in the zone extending to about 4 m (13 ft) below the base of the stone columns has a maximum stress increase of approximately 7%.

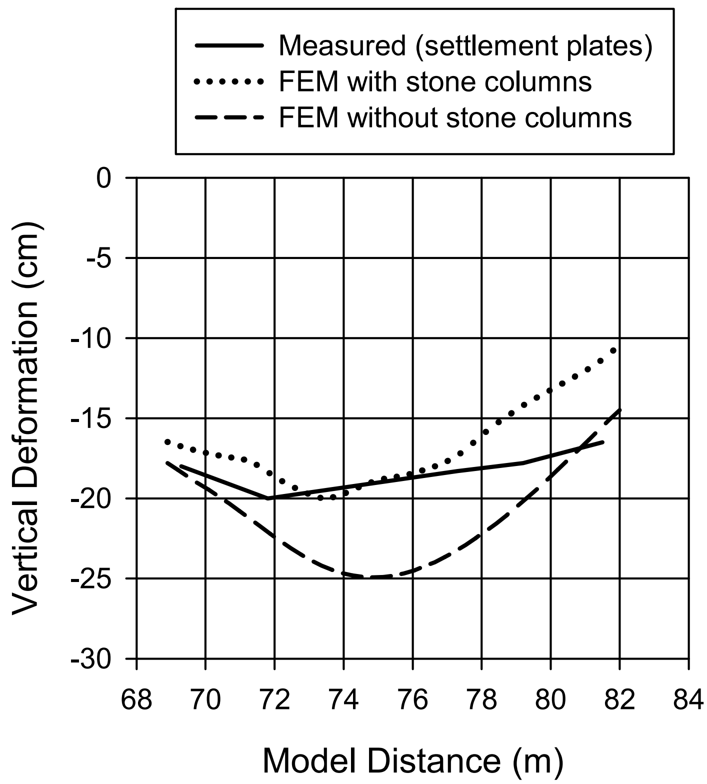

Figure 31. Measured and FEM estimated vertical deformations at the top of the working platform

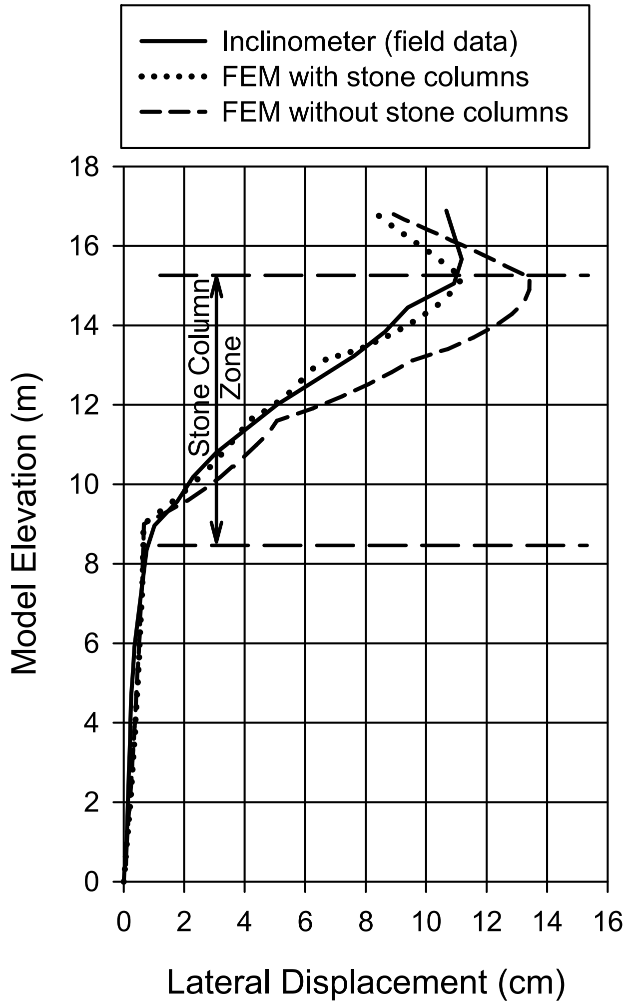

Figure 32. Measured and FEM estimated lateral deformations near toe of widened section

Based on these results, a Boussinesq-type stress distribution can be used to successfully estimate the stress changes in the soil below the reinforced zone. A two-layered elastic system that considers a stiffer upper layer over a lower weaker layer is not required.

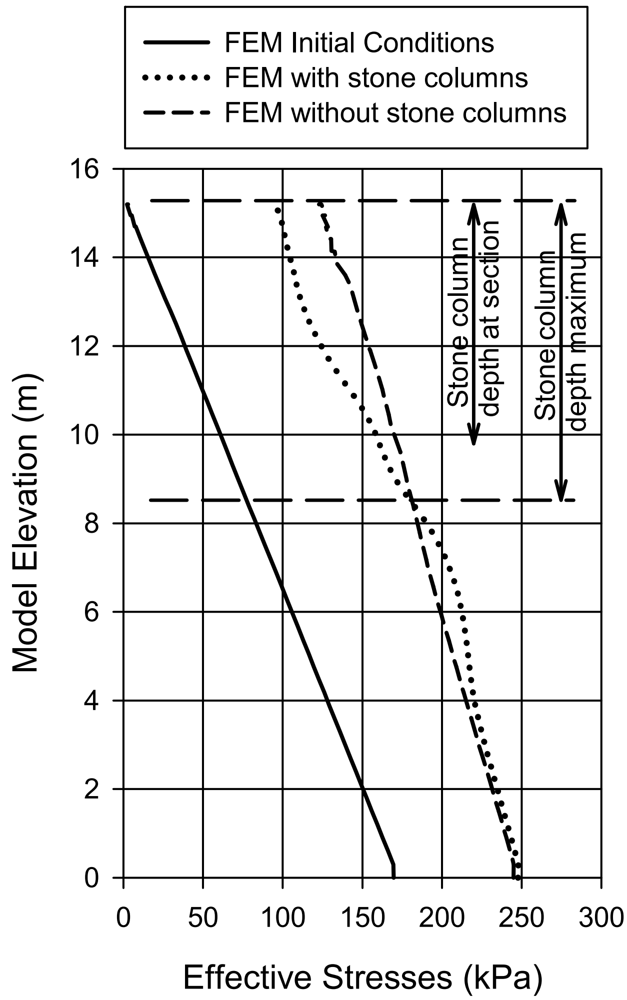

Figure 33. FEM estimated effective stresses at a plane through the soil corresponding to the maximum settlement

Evaluation of Estimated and Measured Settlements

The Equilibrium method requires the SCR in order to complete the estimation procedure. FHWA references suggest values in the range of 2 to 5 (Barksdale and Bachus 1983; Elias et al. 2006). Back-calculation using the Equilibrium method to determine the SCR that would have estimated the measured settlement results in a SCR of 17. At four locations, stresses were measured at the top of stone column and at the soil surface midway between stone columns. Two locations resulted in a SCR of approximately 1.3, and one location resulted in a SCR of 5. One set of instruments did not result in usable data. The FEM model indicated SCRs in the range of 3.5 to 5 for the two center stone columns directly under the full height of the widened section. The measured and modeled SCRs of 1.3 to 5 compared with the back-calculated SCR of 17 to match the measured settlement illustrate the inadequacies of the Equilibrium method. Using SCRs as recommended in the range of 2 to 5 do result in conservative estimates of settlement, which is an advantage of the Equilibrium method.

Based on the measurements from this case history, the Priebe method estimate completed by an experienced stone column contractor closely approximated the measured settlements. McCabe et al. (2009) reported that the Priebe method of estimating settlements is the most favored design approach of leading stone column designers. However, the Priebe method has been shown to not always result in a conservative estimate of settlements. A recent study found the Priebe method to have an approximately 90% probability that the measured settlement will be smaller than the estimated settlement (Douglas and Schaefer 2012).

Practical Considerations and Conclusions

Stone columns have successfully been implemented on transportation-related projects in the U.S. for over three decades. Considerations for future projects as a result of projects with unsatisfactory performance were summarized in Table 28. Deficiencies in three broad areas were identified that contributed to the unsatisfactory performance: site investigation, design, and construction monitoring. In addition to the brief comments in Table 28, considerations for each of these three areas follow, based on the review and evaluation of the case histories.

Site Investigation

As Osterberg (1986) stated, "[T]he realities of foundation engineering are that we never find actual conditions the same as we anticipated." The construction process with repeated insertions of the vibroflot will reveal any deficiencies in the site investigation. For example, the US Highway 42 project had limited borings and geotechnical data. Stiffer soils in the intermediate soft clay layer were encountered during construction than were identified in the investigation. The stiffer soils slowed construction and resulted in a re-design of the project. The future consideration for designers is that a higher level of site investigation is required in order to properly define soil profiles and parameters. The quality of estimates for settlements is directly related to the quality of the site investigation. The site investigation should leave no areas of serious doubt concerning soil conditions, engineering properties, chemical properties, and groundwater conditions (Slocombe 2001).

Design

With regard to estimating settlements and the case history analyzed, the Priebe method is shown to be the preferred method of estimating settlements as compared to the Equilibrium method. However, it should be noted that the Equilibrium method consistently provides a conservative estimate of settlement where the Priebe method has the potential to under-estimate settlements of stone column reinforced ground (Douglas and Schaefer 2012). The portion of settlement attributable to the zone(s) below the bottom of the stone columns can be estimated using traditional consolidation or elastic theory coupled with a traditional

Boussinesq-type stress distribution. Each designer must make a project-specific engineering judgment regarding the depth of the zone to consider in the settlement analysis. Simple elastic finite element models can assist designers in estimating the changes in stress upon loading. Although this paper focused on settlements, every design must also consider bearing capacity and global stability of the structure.

Construction Considerations

No clear, accepted means of anticipating installation effects has been identified, but what is clear is that the installation effects influence the performance of the stone columns (Egan et al. 2009). Installation of stone columns requires experienced operators and close supervision to ensure the design is implemented appropriately in the field. Automated monitoring systems that provide information on the installation process are essential and should be expected from the contractor installing stone columns (Serridge and Synac 2007). The geotechnical engineer of record should be included in the QC/QA program. A specific recommendation for the QC/QA program as a result of the lessons learned is the inclusion of a post-installation geotechnical study to evaluate the installation effects and either confirm or refine the performance estimates.

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CHAPTER 7. GENERAL CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER STUDY

General Conclusions

The papers presented in this dissertation focused on geoconstruction technologies within the field of geotechnical engineering. The information system provides guidance for both well-established and emerging geoconstruction technologies. Application and design guidance for the stone column geoconstruction technology were developed utilizing case histories. The following two sections describe the most important conclusions drawn from the two study areas presented in this dissertation.

Information System

A web-based information and guidance system for 46 geoconstruction technologies was developed and contains an introduction to the geotechnical design process, catalog of technologies, technology selection assistance, and glossary. The information system provides a means for transportation engineers, geologists, planners, and officials; engineering consultants; and others to access state-of-the-practice information for common and emerging geoconstruction technologies available in the U.S. The primary value of the web-based information system is that it collects, synthesizes, integrates, and organizes a vast amount of critically important information about geoconstruction technologies in a system that makes the information readily accessible to transportation agency personnel.

The information system was constructed as a website titled *Geotechnical Solutions for Transportation Infrastructure*. Website development required the integration of engineering and computer science disciplines to produce a dynamic interface that allows users the ability to intuitively and quickly access over 350 technology-specific documents. The dynamic website was developed utilizing Adobe ColdFusion® server software in conjunction with a Microsoft Access® database. The combination of technologies allowed for the various pieces of the information system to be segregated into separate tables within a single database that could be dynamically queried via the web. All aspects of the information system query a database to guide webpage generation and links to the appropriate downloadable files.

A significant outcome of the information system is the Interactive Selection Tool provided as part of technology selection assistance. The Interactive Selection Tool required the formalization of a systematic approach to identifying technologies applicable to the following areas: geosynthetics, geotextiles, ground improvement, grouting, slope stabilization, soil reinforcement, soil stabilization, and alternative/recycled materials. The queries necessary to select a geoconstruction technology were established and coupled to a knowledge base to allow sorting of the geoconstruction technologies. The knowledge base comprises one of the tables in the database. The Interactive Selection Tool dynamically queries the knowledge base to allow a user to efficiently evaluate potential technologies for a specific project. Advanced programming was required to provide the link between the website and the database, to generate subsequent queries based on previous inputs, to sort the list of potential technologies, to control variable values in an internet environment, and to link to a downloadable file to document the results of the system.

Experienced engineers will benefit from the design, construction, cost, and specification information provided in the technology catalog. Less experienced engineers, planners, and owners will benefit from the introductory material for each geoconstruction technology and the technology selection assistance portion of the system to assess the feasibility of technologies to address project requirements and constraints. The information and guidance system will be a valuable tool for engineers, planners, and transportation officials to utilize when evaluating geoconstruction technologies. No system like this existed, either in hard-copy or automated form, prior to the development of the *Geotechnical Solutions for Transportation Infrastructure* system.

Performance of Stone Columns

The performance of stone columns was evaluated through data mining from over 250 literature records that specifically addressed stone columns. Future project considerations were developed from case histories with unsatisfactory performance and from case histories that both estimated and measured settlements.

The stone column case histories identified varied greatly with regard to the information provided for site conditions, soil parameters, design considerations, construction process, and settlement monitoring. The review of case histories revealed the lack of detailed, research-oriented studies that fully document the design, construction, and performance of stone column reinforced ground. Case histories with unsatisfactory performance facilitated the compilation of lessons learned. The case histories with unsatisfactory performance revealed inadequacies in three broad areas that are part of every project: site investigation, design, and construction monitoring. The future project considerations reveal the importance of these three fundamental phases of every geotechnical project. Specific guidance for future projects was developed from these problematic case histories to assist designers in applying, designing, constructing, testing, and monitoring stone columns for future projects.

The performance of stone columns related to reducing settlements was evaluated using both a specific case history and a number of case histories that reported estimated and measured settlements. The Priebe and Equilibrium methods are the two methods discussed in the current FHWA guidance and were evaluated using an embankment widening case history (Elias et al. 2006). The Priebe method was found to be preferred over the Equilibrium method of estimating settlements.

A further evaluation of the Priebe method was completed through a comparison of estimated and measured settlements from case histories. A statistical analysis of the data indicated that the Priebe method provided a conservative estimate of settlements. However, 6 of the 38 data points showed that more settlement was measured than estimated. A reliability framework was utilized to assess the Priebe method. The coefficient of variation (COV) has to be determined in order to complete a reliability assessment. A lower bound COV due to the variability of site conditions was shown to be approximately 20%. An upper bound COV was determined for the estimated-equals-measured line to the Priebe data of approximately 60%. A best-fit line using the reliability approach to fit the Priebe data resulted in a COV of approximately 50%, which corresponded to the Priebe method having an 89% probability that settlements will be smaller than the estimated settlement. Or stated differently, the Priebe method tends to over-estimate measured settlements by approximately 160%.

A consideration for every stone column project is the settlement of the soils below the stone column treated zone. A well-documented case history with measured vertical and horizontal deformations allowed the development of a calibrated finite element model to evaluate the stress distribution below the stone column treated zone. Numerical modeling indicated that the change in stress in the layers underlying the stone column treated ground was approximately 7% higher than the same numerical model without stone columns. The numerical analysis indicated that the stress in zones below the stone column treated ground due to a new surface load can be approximated using a traditional Boussinesq-type stress distribution.

Recommendations for Further Study

Some recommendations for further study for the two topic areas are presented in the following two sections.

Information System

Technologies in the System

The present system was developed for 46 geoconstruction technologies. The addition of other geotechnical construction technologies could broaden the appeal of the system to the geotechnical community. Additional technologies could include bridge and retaining wall foundation systems; deep foundations; shallow foundations; additional earth retaining structures; and other specialty technologies, for example, technologies to address frozen soils, swelling soils, and collapsible soils.

Downloadable Specifications

The usability of the site could be enhanced through the availability of downloadable and editable specification documents for each technology. An initial effort would be to prepare the editable files for each technology. These files could then be incorporated into the system similar to the cost spreadsheets through a link in the products.

Combinations of Technologies

Selection assistance at present leads the user to singular technologies. Combinations of technologies are commonly implemented in practice. Incorporating combinations of complementary technologies could be incorporated in both the technology catalog and within the technology selection assistance.

Regionally Preferred and/or Available Technologies

The system currently does not provide selection based on the location of the project within the U.S. Many regions have commonly preferred solutions that could be incorporated into the system. Additionally, some of the technologies may not be available in some regions or the cost of mobilization to a certain region would make the technology less feasible when compared with other technologies.

Column Supported Embankments Selection Tool

A tool to select the column type for the Column Supported Embankments technology would assist in identifying possible column types for a set of defined project conditions. The tool could sort technologies based on soil conditions, loads, and project serviceability requirements.

Performance-Based, Project-Specific Selection for Geotechnical Pavement Components

Further refinement of the selection assistance in the Geotechnical Pavement Components portion of the Interactive Selection Tool would be beneficial. The refinement should include further demarcation of current technologies that cover many methods and the inclusion of performance based parameters, which could correspond with pavement design inputs. This refinement would also require consideration of combining technologies.

Performance of Stone Columns

Margin of Safety in Settlement Analyses

The standard practice in geotechnical engineering is to explicitly calculate settlements. Future research is needed to determine explicit guidelines regarding acceptable probabilities of exceeding a threshold limit for settlement.

Installation Effects

The installation effects of stone columns are not well documented. The installation effects in different soil types at various strengths with different installation methods require further study. Specifically, the identification of an upper strength limit for the vibro-displacement installation method at which cohesive soils are weakened requires further study.

Instrumented Full-Scale Projects

Further field testing for full-scale projects incorporating stone columns should be conducted. An item-by-item list of information that a study should identify in order to complete an evaluation of the performance of the stone column reinforced ground is presented in Table 30.

Applicability to Embankment Widening

The desire to utilize stone columns to support embankment widening projects will require consideration of differential settlements between the existing and widened sections. Further study is required to determine if stone columns are a viable solution to limit settlements of the existing road when strict serviceability limits are a project requirement.

Table 30. Suggested information for a well-documented case history

Characteristics of a well-documented case history
Soil conditions and variability
Results of field and laboratory testing
Soil parameters used for design
Project conditions and loads used for design
Method utilized for estimating settlements
Estimated settlement as a result of design
Construction process
Design stone column diameter and spacing of stone columns
QA/QC methods and findings during construction
Post-installation soil borings and laboratory testing
Verification of stone column diameter and area ratio
Settlements/heaving (positive or negative elevation change) during stone column construction
Lateral earth pressure changes at various depths and distances from initial to post-installation
Description of construction platform
Load transfer platform over the stone columns
Final loadings as a result of construction
Pressure applied at top of stone columns and surrounding soil
Redundant settlement monitoring, including: • Exact location where settlements were monitored, such as on top of stone column, the soil between the stone columns, or on top of the construction platform/fill/footing. • Settlement of the soils underlying the stone column reinforced ground. • Settlements versus loading, such as monitoring during stage loading.
Pore pressure change with loading and time
Sufficient monitoring program and length of time to define primary consolidation and secondary compression
Extended monitoring time to allow all settlements to cease

APPENDIX A. SUPPLEMENTARY DATA - CASE HISTORY LISTING

Case histories with satisfactory performance have been condensed and sorted according to site conditions and presented in chronological order according to the date of the published reference. Each table provides the reference(s), project and location, soil conditions, and some brief comments regarding the specifics of the case history. The case histories have been sorted by site conditions into different tables as follows:

- Predominately fine-grained soil case histories in Supplementary Table A-1
- Predominately coarse-grained soil case histories to mitigate static settlements in Supplementary Table A-2
- Predominately increasing resistance to liquefaction case histories in Supplementary Table A-3
- Predominately improvement of fill/demolition debris/refuse case histories in Supplementary Table A-4

Case histories applicable to more than one site condition have been included in multiple tables as appropriate. Where multiple references are shown for a specific case history, every paper that describes, provides comments for, or analyzes the case history identified in the search is included in the listing. Complete references for the case histories are provided in the Bibliography.

Supplementary Table A-1. Predominately fine-grained soil settlement case histories

Reference(s)	Project	Soil conditions	Comments
Watt *et al.* 1967; Balaam and Poulos 1983	6 Teesport storage tanks, UK	Soft cohesive soils placed by hydraulic fill	Some center and all perimeter settlements measured in field.
Watt *et al.* 1967; Balaam and Poulos 1983	Hedon storage tank, UK	Soft, natural cohesive soils over firm clays and marls	Perimeter settlements measured in field.
Greenwood 1970; Balaam and Poulos 1983	Bremerhaven road embankment, Germany	Soft peat and clay over fine sands	Test embankment with settlements measured in field.
Greenwood 1974	Multiple storage tanks, UK	Soft estuarine or alluvial soils	Settlement records for 48 tanks, some of which were improved with stone columns.
Engelhardt and Golding 1975; Mitchell and Huber 1983; Barksdale and Bachus 1983a; Mitchell and Huber 1985	Sewage treatment plant, California	Recent fill and estuarine deposits – interbedded clays, silts, and sands.	Settlement and liquefaction improvement.
Hughes *et al.* 1975	Canvey Island column load test, UK	Soft alluvial clay with sand lenses	Tested single column.
Goughnour and Bayuk 1979a; Barksdale and Bachus 1983a; Barksdale and Goughnour 1984	I-64 embankment, Hampton, Virginia	Very soft clay and silt over sand	Well documented test embankment.
Barksdale and Bachus 1983a	River Seine approach embankment, France	Soft clay	Highway embankment, combined with mechanically stabilized wall.
Colleselli *et al.* 1983	Warehouse and 4-story building, Italy	Silty clay and clayey silt, sands, soft organic clay	Combined with vibro-flotation, CPT tests before and after treatment, 3-yr settlement record.
Colleselli *et al.* 1983	Storage tank, Italy	Soft clay over sands	Single column and group load tests, long term perimeter settlement.
Sarkar *et al.* 1983; Barksdale and Goughnour 1984; Munfakh *et al.* 1984; Munfakh 1985	Jourdan Road Terminal, New Orleans port facility, Louisiana	Very soft clay over loose sands and soft sandy clays	Well instrumented test embankment with settlement measurements.
Barksdale and Goughnour 1984;	I-29/US-20 Interchange, Iowa	Loess silty clay underlain by shale bedrock	Instrumented highway embankment, combined with mechanically stabilized wall.

Supplementary Table A-1. (continued)

Reference(s)	Project	Soil conditions	Comments
Greenwood and Kirsch 1984	Bauxite silo, Germany	Clays and silts over marl	Settlement record compared with estimated settlements. Measured settlement of untreated soils below stone columns.
Waterton and Foulsham 1984	Channel Island access road, Australia	Very soft highly plastic silty clay over stiff cohesive soils and sedimentary rock	Specialized installation technique utilizing casing.
Meade and Allen 1985	US 42 Embankment widening, Kentucky	Very soft and soft silts and clays	Instrumented embankment and settlement records during construction.
Litwinowicz and Smith 1988	Gateway arterial, Australia	Very soft clays with sand lenses over mudstone	Overview of incorporation of many ground improvement techniques, some settlement data for stone columns.
Greenwood 1991; Priebe 1995; Ellouze *et al.* 2010	Storage Tank, Canvey Island	Soft clay underlain by silty sand	Measured settlements, loads, and stresses. Settlement record over 175 days.
Greenwood 1991	Humber Bridge South Approach	Soft to stiff silty clay with peat layer underlain by boulder clay and sand	Measured settlements, loads, and stresses. Settlement record over 425 days.
Han and Ye 1991	Coastal Area Field Test, China	Soft silty clay	Detailed plate load tests on unreinforced soil and stone columns.
Jagannatha *et al.* 1991	Ore Handling complex, India	Soft clay over sand	Limited area load tests, rammed column technique.
Slocombe and Mosely 1991	Three projects, UK	Intermixed clays and sands	Three case histories presented with results of automated stone column construction records. Settlement tests presented.
Ergun 1992	Iskenderun Silo, Turkey	Soft clays with compressible peat layers underlain by sands and gravels	Vibro-compaction to 43 ft (13 m) depth and stone columns to 16 ft (5 m). 40-day load and settlement record.
Han and Ye 1993	Storage tank, China	Clayey silt and silty clay	Detailed case history with design and settlement records.
Kundu *et al.* 1994	Storage tank, India	Soft to firm silty clay underlain by very stiff to hard silty clay	Single column load test and perimeter settlement of tank.
Davis and Roux 1997	Rowena water tank, California	Very thin fill underlain by silty clay and clayey silts over sedimentary bedrock	Design to minimize differential settlements on site with varying strength and depth of weak soils.

Supplementary Table A-1. (continued)

Reference(s)	Project	Soil conditions	Comments
Phear 1997; Osbaldeston and Phear 2000	A557 Road at Widnes, UK	Cohesive and granular fill underlain by soft organic clays and firm sandy clay	Design overview with brief discussion of field testing. Load transfer platform with geosynthetic utilized. Contaminated site.
Raju 1997	Shah Alam Expressway, Malaysia	Soft marine clays	Kebun interchange description with 300-day load and settlement data.
Cheung 1998	Auckland Arterial Road, New Zealand	Ash, alluvial silty clays and clayey silts, basalt layers	Instrumented embankment project combined with other geotechnologies; pre-drill and casing installation method.
Venmans 1998	Highway A2, Netherlands	Clays and peats	Instrumented field study of embankment widening on very soft soils. Installation problems observed.
Cooper and Rose 1999	River Avon Bridge Approach, UK	Alluvial silty clays, clayey silts, peat layers, underlain by siltstones, sandstones, and mudstones	Detailed case study with field measurements and settlement profiles beyond area improved.
Manas and Gepp 1999; Samieh 2002	Embankment project, Location unknown	Soft clays	Actual and modeled settlement data.
Clemente and Davie 2000	Test Site 1, Location unknown	Sands underlain by soft silts and clays underlain by sand and coral	Footing load tests with varying stone column spacings including an untreated area.
Watts and Serridge 2000; Watts *et al.* 2001; Serridge and Sarsby 2009	Bothkennar Test Site, UK	Soft clay	Installation influence on column performance, strip load tests, partial depth columns “floating” in soft clay.
Abdrabbo and Mahmoud 2002	Boundary wall, Egypt	Structural fill underlain by soft clay, sand, stiff clay and sandstone	Detailed strip footing design and load test.
Raju 2002	Six highway projects, Malaysia	Very soft silts and clays	Brief summaries of each project with cross-sections and some vertical and lateral deflections.
White *et al.* 2002; Pitt *et al.* 2003	I-35 and IA 5, Iowa	Compressible clay and silt overlying highly weathered shale	Detailed embankment study during construction with settlement data.
Bhushan *et al.* 2004	Two storage tanks, California	Stiff clay fill, soft clays, loose sands, inter-layered clays and sands with depth	Detailed study to reduce settlements in clays and mitigate liquefaction potential in sands; combined with surcharging.

Supplementary Table A-1. (continued)

Reference(s)	Project	Soil conditions	Comments
Oo 2004	Sections 1 and 2, Pantai Expressway, Malaysia	Soft clay underlain by stiff clay	Settlement and rate of consolidation case histories, FEM modeling.
De Silva 2005	Penny's Bay, Hong Kong	Hydraulic sand fill over very soft marine clays and silts, alluvial sands and gravels	Combined stone columns with vibro-compaction and preloading, some settlement records.
Raju and Sondermann 2005; Raju 2010; Yee and Chua 2010	Projects throughout Asia	Mostly weak cohesive soils with some mining slimes	Brief case histories for highways, high speed railways, chemical plants, and airports.
Mestat *et al.* 2006; Wehr and Herle 2006	Well Instrumented Embankment, Location unknown	Thin ancient fill, compressible silty fine grained soils, sandy soils	Results of settlement prediction exercise from a well instrumented test embankment.
Raman 2006	Railroad project, Malaysia	Soft clays and loose sands	Predicted and measured settlements along four sections of alignment.
Lopez and Shao 2007	Costco project, California	Alluvial soils with soft silts and clays with loose sand layers	Static settlement and liquefaction mitigation, construction details.
Bauldry *et al.* 2008	Field House, California	Very soft organic clays underlain by loose sands	Static settlement and liquefaction mitigation, construction details, combined with preloading.
Saroglou *et al.* 2008	New Highway, Greece	Very soft clay with sand and gravel layers	Stability, settlement, and rate of consolidation design summary.
Arulrajah *et al.* 2009	High Speed Railway, Malaysia	Soft clays and loose sands	Design methodology.
Bouassida *et al.* 2009b	Bridge Approach, France to Germany	Soft clay	Project overview with measured settlements.
Wiltafsky and Thurner 2009	Shopping Center, Location Unknown	Soft marine soils, stiff clay, rock	Design and limited monitoring, combined with prefabricated vertical drains and preloading.
Elahi and Sabermahani 2010	Eight-Story Building, Iran	Inter-layered clay, silt and sand	Footing design, construction, and testing.
Raj and Dikshith 2010	Shipyard, India	Some fill, weak marine clay, weathered rock	Design and construction.
Wehr *et al.* 2010	Coal Terminal Expansion, Australia	Very dense sandy fill underlain by very soft clay	Brief overview of project with project settlement requirements and stability design.
Hutchinson 2011	I-44 Intersection, Oklahoma	Clays and clayey silts	Brief project overview, combined with MSEW.

Supplementary Table A-2. Predominately coarse-grained soils, static settlement case histories

Reference(s)	Project	Soil conditions	Comments
Baumann and Bauer 1974	8-story dormitory, Germany	Sands and gravels over varved clay	Footing load tests reported. 3-year settlement record of building.
Rathgeb and Kutzner 1975	Power Plant, Location unknown	Sands with soft silt and gravel layers	Brief project overview.
Munoz and Mattox 1977; Barksdale and Bachus 1983a	Clark Fork Highway, Idaho	Loose sandy silts	Highway embankment project, combined with mechanically stabilized wall.
Bhandari 1983	Storage tank, India	Intermixed sands, silty sands, clayey sands, and sandy clays	Footing load tests provided. Settlements around tank perimeters.
Bell *et al.* 1986	Postal sorting center, Scotland	Thin fill over loose sands	Footing and zone load tests conducted.
Kirsch *et al.* 1986	28 storage tanks, Arabian Peninsula	Loose sands and silts, some underlain by limestone	Estimated and measured settlements for multiple tanks at 3 different projects.
Allen *et al.* 1991	I-90 cut and over tunnel, Washington	Loose gravelly silty sands	Design and settlement of footing to support tunnel wall. Load tests and foundation monitoring.
Hayden and Welch 1991	Naval Air Station Housing, Nevada	Silty sands and sands over highly plastic silt	Increase bearing capacity, reduce settlements, and mitigate liquefaction potential. Detailed load tests.
Hussin and Baez 1991	Building projects Florida, Indiana, Maryland, New Jersey, Texas, and Virginia	Sands and sandy clays	Results of quick load tests to isolated columns to estimate settlements of structures. Modulus of stone column concept.
Watts and Charles 1991	Building foundation test site, UK	Sand with peat layer	Footing load test with settlement record.
Brignoli *et al.* 1994	Ash and gypsum storage area, Italy	Silty sands and sands with silt and clay layers	Well documented tests in treated and untreated areas, special installation method with driven pipe.
Saxena and Saxena 1995	Metro Medical Plaza, Florida	Clayey sands and sandy silts	Improvement verified with CPT testing before and after improvement.
Sondermann 1997	High Speed Line Hanover, Germany	Sands underlain by silts and clays	Overview of stone columns in high speed railway applications.

Supplementary Table A-2. (continued)

Reference(s)	Project	Soil conditions	Comments
Hussin and Musselwhite 1998	Hospital, South Carolina	Interbedded loose sands and soft clays underlain by overconsolidated silt	Static settlement and liquefaction mitigation, test section, footing load tests, CPT verification.
Osborne and Leavy 1999	Residential Development, Australia	Variable thickness loose sands over rock	Design, construction, economics, and CPT testing.
Ashmawy *et al.* 2000	Three projects, Florida	Very loose and loose sands	Plate load tests on single and small groups. Predicted versus measured settlements compared.
Clemente and Davie 2000	Test Site 2, Location unknown	Silts and clays underlain by loose silts and sands underlain by gravelly sands	Footing load tests in treated and untreated areas.
Clemente and Davie 2000	Test Site 4, Location unknown	Carbonate sands underlain by loose sands and silts	Footing load test in treated area.
Slocombe *et al.* 2000	East Anglia	Very silty sands	Brief case history description, CPT before and after treatment.
Slocombe *et al.* 2000	Hartlepool and Heysham Power Plants	Sands	Brief case history description, CPT before and after treatment, lower vibration levels required.
Martinez *et al.* 2001	LNG Tank, Puerto Rico	Thin unengineered fill, marine sands and clays underlain by dense silt sands, limestone	Static settlement and liquefaction concerns, combined with preloading, predicted and measured settlements.
Nnadi *et al.* 2001	Power Plant, Florida	Loose sands underlain at depth by denser sands	General project overview, CPT before and after treatment.
Aiban 2002	Pump House, Saudi Arabia	Loose to medium dense sands	General project overview, design, and construction.
Bouassida *et al.* 2009a; Ellouze *et al.* 2010	Storage Tank, Tunisia	Loose silty sands underlain by marl stone	Project overview with measured settlements.
Blackburn *et al.* 2010	Hospital, New Jersey	Interlayered loose and medium dense sands underlain by dense sands and silts	Static settlement and liquefaction concerns, construction and testing.
Kumar and Ospina 2010	Cruise Berth, Panama	Sand fill, silty and clayey sands, weathered rock	Brief overview of project.
Wehr *et al.* 2010	High Speed Railway, Germany	Fine to medium sands	Brief overview of project.
Wehr *et al.* 2010	Two LNG Tanks, India	Silty fine sands	Brief overview of project.

Supplementary Table A-3. Predominately increasing resistance to liquefaction case histories

Reference(s)	Project	Soil conditions	Comments
Engelhardt and Golding 1975; Mitchell and Huber 1983; Barksdale and Bachus 1983a; Mitchell and Huber 1985	Sewage treatment plant, California	Recent fill and estuarine deposits - interbedded clays, silts, and sands	Settlement and liquefaction improvement.
Glover 1985	Industrial complex, Malaysia	Coral sand hydraulic fill with some clay and silt layers	Increase bearing capacity and resistance to liquefaction.
Hayden and Welch 1991	Naval Air Station Housing, Nevada	Silty sands and sands over highly plastic silt	Increase bearing capacity, reduce settlements, and mitigate liquefaction potential. Before and after CPT testing.
Egan *et al.* 1992; Elias *et al.* 2006	7th Street Terminal, California	Sand fills	Liquefaction retrofit project.
Ergun 1992	Iskenderun Silo, Turkey	Soft clay with compressible peat layers underlain by sands and gravels	Vibro-compaction to 43 ft (13 m) depth and stone columns to 5 m. CPT before and after treatment.
Allen *et al.* 1995; Kelsic *et al.* 1995; Rollins and Giles 2002	Mormon Island Auxiliary Dam, California	Dredged sands, gravels, and cobbles	Construction process with before and after shear wave velocity profiles.
Swenson *et al.* 1995	Mariner Square Bulkhead, California	Hydraulically placed, loose sands	Repair and remediation project, SPT before and after improvement.
Yourman *et al.* 1995	Terminal Island Structures, California	Hydraulic fill sands and silty sands underlain by alluvial interbedded sands and silty sands	Design and construction details. Variable spacing test sections with SPT and CPT before and after improvement.
Somasundaram *et al.* 1997	Long Beach Aquarium, California	Fills, hydraulic fills, and native deposits consisting of sands, silty sands, sandy and clayey silts, and clays	Detailed analysis, pilot test program, construction, and CPT verification.
Soydemir *et al.* 1997	Albany County Airport, New York	Sands and gravels	Detailed analysis, test program, construction Quality Control, and CPT and SPT verification.
Hussin and Musselwhite 1998	Hospital, South Carolina	Interbedded loose sands and soft clays underlain by overconsolidated silt	Static settlement and liquefaction mitigation, test section, footing load tests, and CPT verification.

Supplementary Table A-3. (continued)

Reference(s)	Project	Soil conditions	Comments
Ashford *et al.* 2000	Treasure Island Liquefaction Test, California	Interlayered loose sands and soft clays	Full-scale lateral load tests on cast-in-steel-shell piles before and after installation of stone columns around piles.
Martinez *et al.* 2001	LNG Tank, Puerto Rico	Thin unengineered fill, marine sands and clays underlain by dense silt sands, limestone	Static settlement and liquefaction concerns, combined with preloading, predicted and measured settlements.
Brunner *et al.* 2002	Formosa Plant, Taiwan	Hydraulic sands	Brief design and construction, post-earthquake settlements, combined with deep compaction.
Maduro *et al.* 2004	Coco Beach Resort, Puerto Rico	Sand fill, weak swamp deposits consisting of silt, sand and peat	Brief project overview, settlement and liquefaction considerations.
Bhushan *et al.* 2004	Two storage tanks, California	Stiff clay fill, soft clays, loose sands, interlayered clays and sands with depth	Detailed study to reduce settlements and mitigate liquefaction potential; combined with surcharging.
Chen and Bailey 2004	Seattle Embankment, Washington	Interlayered alluvial sand, silt, clay and occasional peat underlain by sands and gravels	Test program to reduce settlements and liquefaction potential, CPT testing before and after improvement.
Vrettos and Savidis 2004	Highway Tunnel, Greece	Irregular layers of sands, silts, and clays	Detailed seismic evaluation, liquefaction susceptibility, and design.
Wijewickreme and Atukorala 2005	Natural Gas Pipeline Station, British Columbia	Loose sands and sandy silts, soft clays, denser sands with depth	Site assessment and evaluation, construction, and CPT testing.
Wijewickreme and Atukorala 2005	Trans Canada Highway, British Columbia	Sands and gravels	Safety level retrofit to minimize bridge collapse, rather than functionality. Detailed design and construction monitoring.
Ausilio and Conte 2007	Village Reconstruction, Italy	Silty soils with interbedded gravel layers	Post-earthquake improvement prior to reconstruction. Testing before and after treatment.
Lopez and Shao 2007	Costco project, California	Alluvial soils with soft silts and clays with loose sand layers	Settlement and liquefaction mitigation design and construction.
Bauldry *et al.* 2008	Field House, California	Very soft organic clays underlain by loose sands	Static settlement and liquefaction mitigation, construction details, combined with preloading.
Arman *et al.* 2009	Seismic Retrofit, Turkey	Deep alluvial deposits consisting of clayey silt, sandy clay, and sand	Seismic retrofit of buildings, modified installation procedure.

Supplementary Table A-3. (continued)

Reference(s)	Project	Soil conditions	Comments
Rollins *et al.* 2009	Ogden Test Site, Utah	Silty sands and sandy silts	Field test with and without prefabricated vertical drains, SPT testing before and after improvement.
Shao 2009	Home Depot, California	Interbedded sand and clays of varying strengths	Project overview, combined with deep soil mixing columns.
Blackburn *et al.* 2010	Hospital, New Jersey	Interlayered loose and medium dense sands underlain by dense sands and silts	Static settlement and liquefaction concerns, construction and testing.
Kumar and Ospina 2010	Cruise Berth, Panama	Sand fill, silty and clayey sands, weathered rock	Brief overview of project.

Supplementary Table A-4. Predominately improvement of fill/demolition debris/refuse case histories

Reference(s)	Project	Soil conditions	Comments
Greenwood 1970	Silo Foundations, UK	City refuse and general dump	Provides results of plate bearing tests at 7 sites with various types of fill.
Glover 1985	Industrial complex, Malaysia	Coral sand hydraulic fill with some clay and silt layers	Increase bearing capacity and resistance to liquefaction.
Slocombe 1989	Distribution warehouse, UK	Variable fill, alluvium, boulder clay	Overview of application to difficult, inner city site.
Callanan 1991	Six storage tanks, Ireland	Hydraulic fill consisting of loose silty gravelly sand over soft estuarine silt and clay	Design and construction details with some limited field settlement data, SPT data before and after treatment.
Davie *et al.* 1991	Gilberton Power Plant, Pennsylvania	Culm fill (coal waste) over sandy clay and silty sand	Design, layout, installation problems, performance assessment, and plate bearing test.
Greenwood 1991	St. Helens	Granular fill	Discussion of load test of strip footing on stone column.
Snethen and Homan 1991	State Highway 11, Oklahoma	Uncontrolled fill and trash	Stone columns utilized after deep dynamic compaction.
Watts and Charles 1991	Building foundation test site, UK	Miscellaneous clay fill in former gravel pit	Footing load test with settlement record.

Supplementary Table A-4. (continued)

Reference(s)	Project	Soil conditions	Comments
Buggy *et al.* 1994	Two storage tanks, Florida	Hydraulic fill consisting of sands and clays underlain by silty clay, sand, and limestone	Design, construction, and settlement data presented. Finite element model to predict settlements.
Allen *et al.* 1995; Kelsic *et al.* 1995; Rollins and Giles 2002	Mormon Island Auxiliary Dam, California	Dredged sands, gravels, and cobbles	Construction process with before and after shear wave velocity profiles.
Swenson *et al.* 1995	Mariner Square Bulkhead, California	Hydraulically placed, loose sands	Liquefaction repair and remediation project.
Yourman *et al.* 1995	Terminal Island Structures, California	Hydraulic fill sands and silty sands underlain by alluvial interbedded sands and silty sands	Liquefaction potential mitigation. Variable spacing test sections with SPT and CPT before and after improvement.
Raju 1997	Shah Alam Expressway, Kuala Lumpur	Tin mining slime	Kinrara interchange description with 300-day load and settlement data.
Saxena and Hussin 1997	Building Complex, Florida	Dredged sand fill underlain by sandy peats, silty sands, limerock	Design, construction, settlement, and CPT testing for buildings up to 6-stories high.
Somasundaram *et al.* 1997	Long Beach Aquarium, California	Fills, hydraulic fills, and native deposits consisting of sands, silty sands, sandy and clayey silts, and clays	Detailed liquefaction analysis, pilot test program, construction, and CPT verification.
Clemente and Davie 2000; Clemente and Parks 2005	Test Site 3 (2000), referred to as Power Station, UK (2005)	Heterogeneous fill with sand, sandy clay, brick fragments, ash, and concrete underlain by alluvial sands and glacial sands	Footing load tests in treated and untreated areas.
Renton-Rose *et al.* 2000	Coke Calcining Plant, Bahrain	Sea dredged sand and gravel underlain by marine sands and sandstone	Design information with plate bearing test results.
Slocombe *et al.* 2000	LNG Plant, Trinidad	Hydraulic fill with equal amounts of very silty sand and cohesive soils	Brief case history description, CPT before and after treatment.
Watts *et al.* 2000	Trial embankment, UK	Variable ash fill and clay fill underlain by glacial till	Detailed test of strip footings, lateral stress increase during installation.
Brunner *et al.* 2002	Formosa Plant, Taiwan	Hydraulic sands	Brief design and construction, post-earthquake settlements, combined with deep compaction.
Taube and Herridge 2002	Storage Tank, Pennsylvania	Industrial fill including silt, glass, brick fragments, cinders, slag, and coal underlain by silts and gravels	Brief design and construction with settlements during hydrotesting.

Supplementary Table A-4. (continued)

Reference(s)	Project	Soil conditions	Comments
Maduro *et al.* 2004	Coco Beach Resort, Puerto Rico	Sand fill, weak swamp deposits consisting of silt, sand and peat	Brief project overview, settlement and liquefaction considerations.
Raju *et al.* 2004	Kajang Ring Road, Malaysia	Tin mining slime submerged under a pond	Detailed project overview with construction and 500-day settlement data.
Wilder *et al.* 2008	Trenton Water Treatment Facility, New Jersey	Existing fill and soft materials	Project overview with load testing.
Sharma and Sapkota 2009	Desalination Plant, Algeria	Construction debris consisting of clays, sands, gravels, wood pieces, bricks, and concrete, over a marl bedrock	Brownfield site, design, construction, and load testing.

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