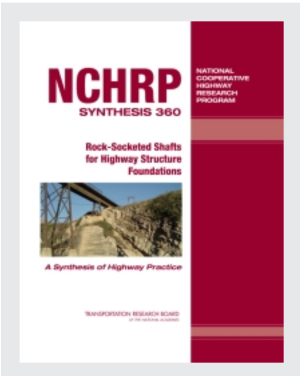

Rock-Socketed Shafts for Highway Structure Foundations

DETAILS

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP SYNTHESIS 360

Rock-Socketed Shafts for Highway Structure Foundations

A Synthesis of Highway Practice

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Laramie, Wyoming

SUBJECT AREAS
Bridges, Other Structures, and Hydraulics and Hydrology and Soils, Geology, and Foundations

Research Sponsored by the American Association of State Highway and Transportation Officials
in Cooperation with the Federal Highway Administration

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Cover photograph: Bridge with surface exposures of foundation rock.

FOREWORD

By Staff
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Highway administrators, engineers, and researchers often face problems for which information already exists, either in documented form or as undocumented experience and practice. This information may be fragmented, scattered, and unevaluated. As a consequence, full knowledge of what has been learned about a problem may not be brought to bear on its solution. Costly research findings may go unused, valuable experience may be overlooked, and due consideration may not be given to recommended practices for solving or alleviating the problem.

There is information on nearly every subject of concern to highway administrators and engineers. Much of it derives from research or from the work of practitioners faced with problems in their day-to-day work. To provide a systematic means for assembling and evaluating such useful information and to make it available to the entire highway community, the American Association of State Highway and Transportation Officials—through the mechanism of the National Cooperative Highway Research Program—authorized the Transportation Research Board to undertake a continuing study. This study, NCHRP Project 20-5, "Synthesis of Information Related to Highway Problems," searches out and synthesizes useful knowledge from all available sources and prepares concise, documented reports on specific topics. Reports from this endeavor constitute an NCHRP report series, *Synthesis of Highway Practice.*

This synthesis series reports on current knowledge and practice, in a compact format, without the detailed directions usually found in handbooks or design manuals. Each report in the series provides a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems.

PREFACE

During the past 25 years, much knowledge and experience has been acquired by the engineering and construction industries on the use of rock-socketed shafts for support of transportation structures. This synthesis collected, reviewed, and organized the most salient aspects of this knowledge and experience to present it in a form useful to foundation designers, researchers, contractors, and transportation officials. The objectives of this report were to collect and summarize information on current practices pertaining to each step of the design process, along with the limitations; identify emerging and promising technologies; determine the principal challenges in advancing the state of the practice; and provide suggestions for future developments and improvements in the use and design of rock-socketed shafts.

For this TRB synthesis report a literature review was conducted on all topics related to drilled shaft in rock or intermediate geomaterials. A questionnaire was developed and distributed to the principal geotechnical and structural engineers of U.S. state and Canadian provincial transportation agencies. Questions were grouped into the following categories: use of rock-socketed shafts by the agency, evaluation of rock and intermediate geomaterials, design methods for axial loading, design methods for lateral loading, structural design, construction, and field load and integrity testing.

John Turner, Professor of Civil and Architectural Engineering, University of Wyoming, Laramie, collected and synthesized the information and wrote the report, under the guidance of a panel of experts in the subject area. The members of the topic panel are acknowledged on the preceding page. This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As progress in research and practice continues, new knowledge will be added to that now at hand.

CONTENTS

1 SUMMARY

3 CHAPTER ONE INTRODUCTION
Background, 3
Problem Definition, 3
Scope and Objectives, 3
Methodology, 4
Organization of Synthesis, 4
Design Process, 5

8 CHAPTER TWO SITE AND GEOMATERIAL CHARACTERIZATION
Scope, 8
Site Geology, 8
Field Investigations, 10
Geologic and Index Properties of Rock, 16
Engineering Properties of Rock, 20
Intermediate Geomaterials, 29
Summary, 31

33 CHAPTER THREE DESIGN FOR AXIAL LOADING
Scope, 33
Relationship to Geomaterial Characterization, 33
Load Transfer Behavior of Rock Sockets, 33
Capacity Under Axial Loading, 37
Axial Load-Displacement Behavior, 47
Current AASHTO Practice, 51
Summary, 52

54 CHAPTER FOUR DESIGN FOR LATERAL LOADING
Scope, 54
Design Process, 54
Rock-Socketed Foundations for Lateral Loading, 54
Analytical Methods, 55
Structural Issues, 66
Summary, 70

71 CHAPTER FIVE CONSTRUCTION AND FIELD TESTING
Scope, 71
Construction of Rock Sockets, 71
Field Load Testing, 74
Constructability, Inspection, and Quality Assurance, 88
Examples of Difficult Geologic Conditions, 93
Summary, 95

97 CHAPTER SIX CONCLUSIONS
Site and Geomaterial Characterization, 97
Design for Axial Load, 98
Design for Lateral Load, 99
Load Testing, 99
Constructability and Inspection, 99
Research Needed to Advance State of Practice, 100

102 EQUATION SYMBOLS

104 REFERENCES

110 APPENDIX A SURVEY RESPONDENTS

111 APPENDIX B SURVEY QUESTIONNAIRE AND RESPONSES

ROCK-SOCKETED SHAFTS FOR HIGHWAY STRUCTURE FOUNDATIONS

SUMMARY Drilled shafts are one of the few structural foundation types that can be built directly into rock. Foundations in rock are attractive because high load carrying capacities are possible and foundation displacements can be limited to acceptable levels more readily than through foundations in soil. Over the past 25 years, much knowledge and experience has been gained by the engineering and construction industries with the use of rock-socketed drilled shafts for support of transportation structures. The goal of this synthesis is to collect, review, and organize the most salient aspects of that knowledge and experience and to present it in a form that is useful to foundation designers, researchers, contractors, and transportation officials.

Challenges faced by foundation designers when considering rock-socketed drilled shafts include: (1) characterizing the nature of the rock mass or intermediate geomaterial, (2) selecting appropriate design methods for analysis of axial load carrying capacity and axial load-deformation response, (3) analysis and design for lateral loading, and (4) assessing issues of constructability and their influence on foundation performance and costs. Each of these issues is considered in the synthesis within the context of the overall foundation design process as practiced by transportation agencies.

A survey questionnaire was developed and distributed to the principal geotechnical and structural engineers of 52 U.S. transportation agencies (including Puerto Rico and the District of Columbia) and Canadian provinces. The purpose of the survey was to define the current state of practice for rock-socketed drilled shafts. Thirty-two U.S. transportation agencies and one Canadian provincial transportation agency responded to the questionnaire.

Innovative methods for field load testing of drilled shafts, including the Osterberg Cell and Statnamic methods, have contributed to advances in design and construction of shafts in rock. Load testing is shown to be an integral part of several state department of transportation programs that have led to increased use of rock-socketed drilled shafts and improved design methods. These and other load testing methods for rock-socketed shafts are reviewed.

CHAPTER ONE

INTRODUCTION

BACKGROUND

Highway bridges represent a large investment in the U.S. transportation infrastructure, and structural foundations account for a significant percentage of total bridge costs. Current foundation engineering practice in the transportation industry represents a dramatic advancement compared with 25 years ago. Development of this topic is illustrated by considering that *NCHRP Synthesis 42: Design of Pile Foundations* (Vesic 1977) does not mention rock-socketed drilled shafts. At the time of its publication, *NCHRP Synthesis 42* was the most comprehensive study extant on the use of deep foundations for transportation structures. According to DiMaggio (2004), in 1980, driven piles accounted for more than 95% of transportation market share, based purely on repeating previous practice. Today, the practice is oriented toward matching the foundation type to project conditions. This has led to a wider variety of deep foundation types selected on the basis of subsurface conditions, structural behavior, constructability, environmental constraints, and cost. A foundation type that has steadily increased in use over this time is the drilled shaft, a deep foundation constructed by placing fluid concrete in a drilled hole.

A potentially effective way to use a drilled shaft is by bearing on, or extending into, rock. To achieve the performance and economy potentials of rock-socketed shafts, designers must be aware of the many issues that affect both cost and performance. Drilling and excavation in rock is generally more expensive and time consuming than in soil. Construction of a rock socket poses challenges and difficulties that are unique and may require specialized techniques, equipment, and experience. The first issue confronted by a foundation designer is to determine whether a rock-socketed foundation is necessary for bridge support. Factors to consider include the nature and magnitude of structural loads and factors related to rock mass characteristics, including depth to rock, rock type, rock mass engineering properties, and constructability. The additional costs and effort of construction in rock must be offset by its benefits. The principal benefits normally are higher load-carrying capacity and the ability to limit deformations, compared with foundations not founded in rock. To make the appropriate cost comparisons, rock-socket design must be based on rational models of behavior that reliably predict the capacity and load-deformation behavior.

PROBLEM DEFINITION

The engineering problem addressed by this synthesis is shown in Figure 1. A drilled shaft foundation is to be designed and constructed for support of a bridge structure. Subsurface conditions may consist of soil underlain by rock. Upper portions of the rock may be partially to highly weathered, giving these materials engineering properties that are transitional between soil and rock, sometimes referred to as intermediate geomaterials, or IGM. Loads to be considered for design typically are determined by AASHTO *Bridge Design Specifications*, with proper consideration of load combinations and load factors. For foundation analysis, design loads may be resolved into vertical (P), horizontal (H), and moment (M) components at the head of the shaft. A subsurface investigation is required to provide information on all of the geomaterials through which the shaft must be constructed and from which the foundation will derive its resistance to the design loads. The foundation designer then must determine the required dimensions (depth and diameter) and structural properties of drilled shafts that will provide adequate resistance and will limit vertical and horizontal deformations to a level that provides adequate service performance of the bridge. Trial designs are developed and evaluated with respect to: (1) cost, (2) performance, and (3) constructability. A major factor in all three criteria is whether the shaft needs to be extended into the rock or IGM layers. Rock sockets will generally increase costs, improve load-carrying and load-displacement performance, and make construction more challenging.

SCOPE AND OBJECTIVES

The overall objectives of this synthesis study are to

- Collect and summarize information on current practices pertaining to each step of the process described previously, along with their limitations and sources of uncertainty;
- Identify emerging and promising technologies in each of these areas;
- Identify the principal challenges in advancing the state of the practice; and
- Provide suggestions for future developments and improvements in the use and design of rock-socketed shafts.

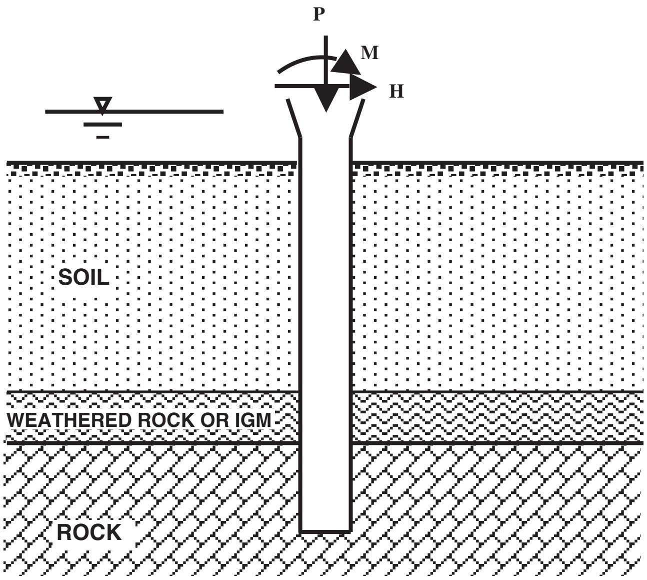


FIGURE 1 Rock-socketed shaft designed for highway bridge structure.

The major challenges faced by U.S. transportation agencies in the use of rock-socketed drilled shafts for highway bridges were identified by NCHRP Topic Panel 36-12 as follows:

- The first challenge is characterizing the nature of the rock or IGM. By its nature, rock and IGM are highly variable and difficult to characterize for engineering purposes. To effectively design drilled shafts in rock and IGM, engineers must accommodate high levels of uncertainty. Issues to be addressed include quantifying material characteristics, rock mass behavior, and appropriate application of laboratory and field test methods.
- The second challenge is determination of the axial load capacity of rock-socketed shafts. Rock-socketed shafts resist axial load in both side shear and end bearing. Designers need well-documented methods for assessing side shear and end bearing. Different methods are appropriate for different types of geology. There are many issues related to characterizing the rock and construction that affect design for axial loading.
- The third challenge is analysis and design of rock-socketed shafts under lateral loading. It has been a customary practice to adopt the techniques developed for laterally loaded piles in soil to solve the problem of rock-socketed shafts under lateral loading. There exist several analysis and design methods specifically for rock-socketed shafts under lateral loading; their application in practice remains limited.

METHODOLOGY

A literature review was conducted on all topics related to drilled shafts in rock or IGM. To assess current practice, the primary manuals used by transportation engineers for drilled shaft design were consulted. These include *Drilled Shafts: Construction Procedures and Design Methods* by O'Neill and Reese (1999) and the AASHTO *LRFD Bridge Design Specifications* (3rd ed. 2004). In addition, a draft version of Section 10, "Foundations," of the 2006 Interim AASHTO *LRFD Bridge Design Specifications* was reviewed.

A questionnaire was developed and sent to the principal geotechnical and structural engineers of 52 U.S. transportation agencies (including Puerto Rico and the District of Columbia) and the Canadian provinces. The primary purpose of the survey was to define the current state of practice for rock-socketed shafts. Questions were grouped into the following categories:

- Use of rock-socketed shafts by the agency,
- Evaluation of rock and IGM properties,
- Design methods for axial loading,
- Design methods for lateral loading,
- Structural design,
- Construction, and
- Field load and integrity testing.

Thirty-two U.S. and one Canadian provincial transportation agencies responded to the questionnaire, completely or in part. A list of responding agencies and a summary of responses to the questions are given in Appendix A. The questionnaire was also sent to several consulting firms and drilled shaft contractors. Two contractors responded to the survey. Based on responses to the questionnaire, selected state agency personnel and contractors were interviewed.

ORGANIZATION OF SYNTHESIS

The synthesis is presented in six chapters and two appendixes. Chapter one defines the problem, objectives, scope, and methodology of the study. This chapter also provides an overview of the foundation design process used by state department of transportation (DOT) agencies. This overview provides a framework for understanding the interrelationships between site characterization, material property evaluation, geotechnical and structural design, load testing, and construction of rock-socketed shafts. Each of these topics is considered in subsequent chapters. Chapter two reviews methods of site characterization and material property evaluation that are applicable to rock-socketed shafts. Chapter three is a compilation and critical review of methods used for analysis and design of rock sockets for axial loading. Chapter four reviews and summarizes analysis methods for rock sockets under lateral and moment loading and discusses structural design issues relevant to rock sockets. Chapter five provides an overview of current technologies for rock-socket construction and considers some of the construction issues identified by the survey. This chapter also covers field load testing of rock-socketed shafts and the role of load testing within the context of state DOT foundation engineering programs. Chapter six is a summary of the principal findings of this study and presents steps that can lead to more effective use, design, and construction of rock sockets for bridge foundations. In each chapter, significant findings derived from the survey are identified and discussed. Appendix A provides a list of survey respondents and Appendix B presents the questionnaire and a compilation of the responses to each question.

DESIGN PROCESS

Structural foundation design within state DOTs is typically a joint effort between the structural and geotechnical divisions. The geotechnical group may include engineering geologists and both groups may operate under the supervision of a chief bridge engineer. As a starting point, consider Figure 2, which shows a flow chart of the overall foundation design process. The chart is from the Washington State DOT *Geotechnical Design Manual* (2005). Based on responses to Question 4 of the survey questionnaire (Appendix B) and interviews with DOT personnel, Figure 2 typifies the process followed by many states. A summary of each step, also based on the Washington State DOT manual (2005), is as follows.

Conceptual Bridge Foundation Design

An informal communication/report is produced by the Geotechnical Division (GD) at the request of the Bridge and Structures Office (Bridge). Information provided includes a brief description of the anticipated site conditions, conceptual foundation types considered to be feasible, and conceptual evaluation of potential geotechnical hazards such as liquefaction. The purpose of these recommendations is to provide sufficient geotechnical information to allow a bridge preliminary plan to be produced.

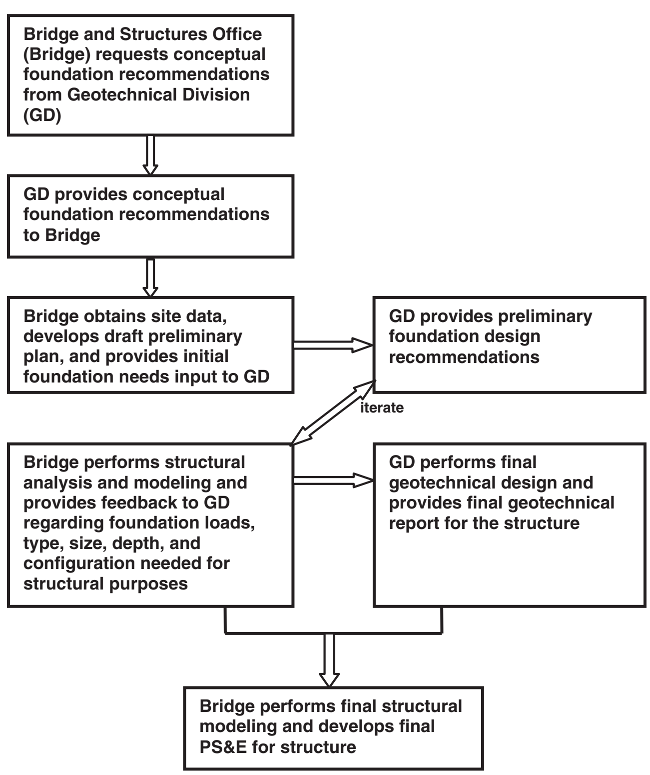


FIGURE 2 Design process for Load and Resistance Factor Design (Washington State DOT 2005). PS&E = Plans, Specifications, & Estimates.

Develop Site Data and Preliminary Plan

Bridge obtains site data from the regional office and develops a preliminary bridge plan (or other structure) adequate for GD to locate borings in preparation for final design of the structure. Bridge would also provide the following information to GD to support development of the preliminary foundation design:

- Anticipated structure type and magnitudes of tolerable settlement (total and differential).
- At abutments, the approximate maximum elevation feasible for the top of the foundation.
- For interior piers, the number of columns anticipated and, if there will be single foundation elements for each column or if one foundation element will support multiple columns.
- At stream crossings, the depth of scour anticipated, if known. Typically, GD will pursue this issue with the Hydraulics Office.
- Known constraints that would affect the foundations in terms of type, location, or size, or any known constraints that would affect the assumptions made to determine the nominal resistance of the foundation (e.g., utilities that must remain, construction staging needs, excavation, shoring and falsework needs, and other constructability issues).

Preliminary Foundation Design

A memorandum is produced by GD at the request of Bridge that provides geotechnical data adequate to conduct structural analysis and modeling for all load groups to be considered. The geotechnical data are preliminary and not in final form for publication and transmittal to potential bidders. At this stage, foundation recommendations are subject to change, depending on the results of structural analysis and modeling and the effect that modeling and analysis has on foundation types, locations, sizes, and depths, as well as any design assumptions made by the geotechnical designer. Preliminary foundation recommendations may also be subject to change based on construction staging needs and other constructability issues discovered during this phase. Geotechnical work conducted during this stage typically includes completion of the field exploration program to the final PS&E level (Plans, Specifications, & Estimates), development of foundation types and feasible capacities, foundation depths needed, *p-y* curve data and soil spring data for seismic modeling, seismic site characterization and estimated ground acceleration, and recommendations to address known constructability issues. A description of subsurface conditions and a preliminary subsurface profile would also be provided at this stage; however, detailed boring logs and laboratory test data would usually not be provided.

Structural Analysis and Modeling

Bridge uses the preliminary foundation design recommendations provided by GD to perform structural modeling of the foundation system and superstructure. Through this modeling, Bridge determines and distributes the loads within the structure for all appropriate load cases, factors the loads as appropriate, and sizes the foundations using foundation nominal resistances and resistance factors provided by GD. Constructability and construction staging needs continue to be investigated during this phase. Bridge provides the following feedback to GD to allow them to check their preliminary foundation design and produce the Final Geotechnical Report for the structure:

- Anticipated foundation loads (including load factors and load groups used),
- Foundation size/diameter and depth required to meet structural needs,
- Foundation details that could affect the geotechnical design of the foundations, and
- Size and configuration of deep foundation groups.

Final Foundation Design

This design step results in a formal report produced by GD that provides final geotechnical recommendations for the subject structure. This report includes all geotechnical data obtained at the site, including final boring logs, subsurface profiles, laboratory test data, all final foundation recommendations, and final constructability recommendations for the structure. At this time, GD checks the preliminary foundation design in consideration of the structural foundation design results determined by Bridge, and makes modifications as needed to accommodate the structural design needs provided by Bridge. Some state DOTs may also make this report available to potential bidders.

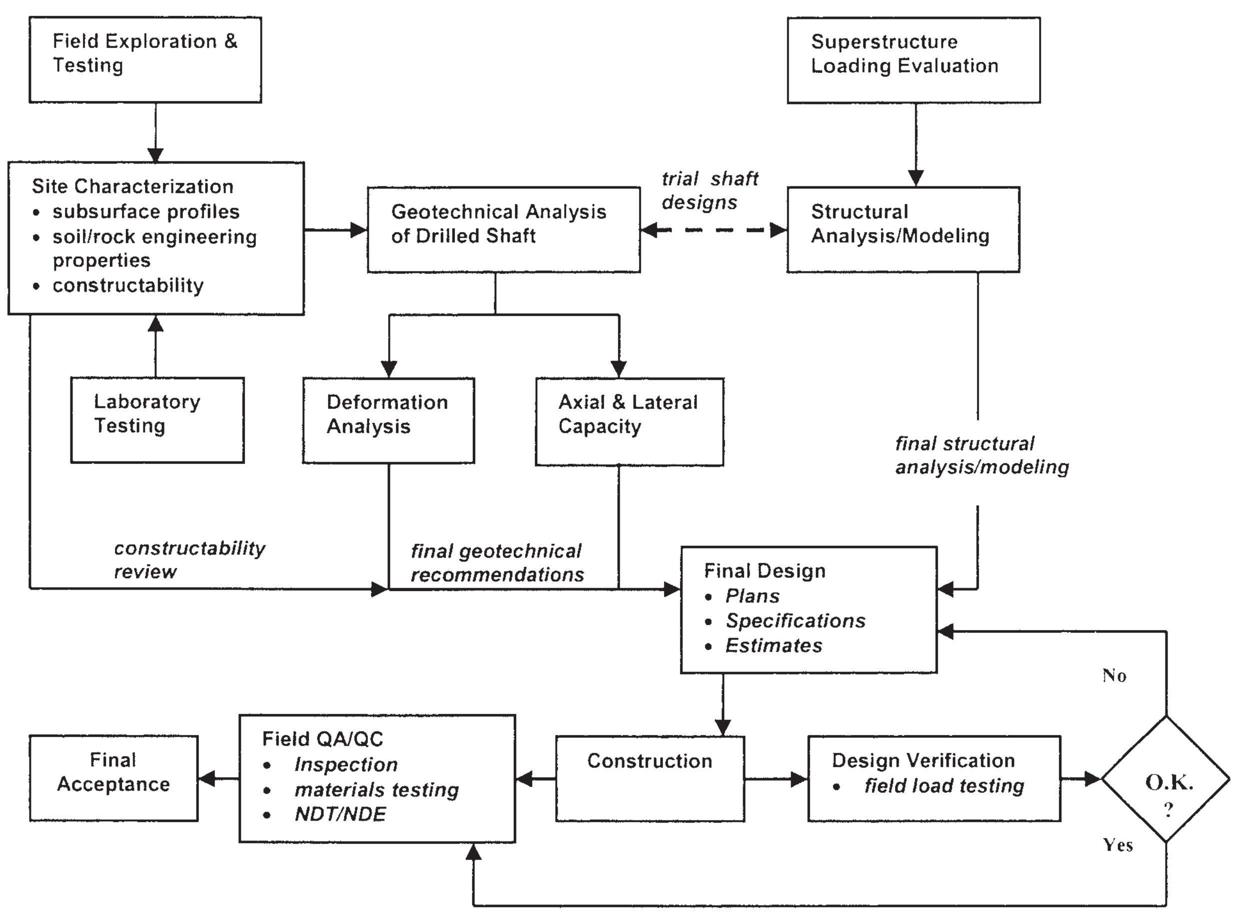


FIGURE 3 Design and construction process for drilled shaft foundations (adapted from Paikowsky et al. 2004a). QA/QC = quality assurance/quality control; NDT/NDE = nondestructive testing/evaluations.

Final Structural Modeling and Development of Plans, Specifications, and Estimates

Bridge makes the required adjustments to the structural model to accommodate changes in the geotechnical foundation recommendations as transmitted in the final geotechnical report. From this, the bridge design and final PS&E are completed. A similar design process is recommended if a consultant or design–builder is performing one or both design functions.

Design Process in Relation to the Synthesis

Based on the process described previously and followed by most state DOTs, Figure 3 is a flowchart of the design and construction process for drilled shaft foundations that provides a framework for the topics addressed by this synthesis. In each subsequent chapter, the topics being covered are considered within the context of the overall process as shown in Figure 3. This includes site investigation, geomaterial property evaluation, and design for axial and lateral loading.

CHAPTER TWO

SITE AND GEOMATERIAL CHARACTERIZATION

SCOPE

This chapter describes site investigation methods, classification systems for intact rock and rock masses, and field and laboratory tests used to determine rock engineering properties. The focus is limited to information relevant to the design and construction of rock-socketed drilled shafts. Several references are available that provide guidance on strategies and methods of site characterization and material property evaluation for geotechnical practice, with a focus on transportation facilities. These include the FHWA *Manual on Subsurface Investigations* (Mayne et al. 2001), "Evaluation of Soil and Rock Properties," *Geotechnical Engineering Circular No. 5* (Sabatini et al. 2002), and the AASHTO *Manual on Subsurface Investigations* (1988). In addition, the U.S. Army Corps of Engineers has published several manuals relevant to this topic (*Rock Testing Handbook* 1993; *Rock Foundations* 1994; "Geotechnical Investigations" 2001).

The purpose of site characterization is to obtain the information required to develop a model of the site geology and to establish the required engineering properties of the geomaterials. The information obtained is used for two general purposes: (1) analysis of capacity and load-deformation response, which determines the foundation overall design; and (2) construction feasibility, costs, and planning. Once the site for a bridge or other transportation structure has been established, all aspects of the site and material characterization program are focused on the soil and rock conditions as they exist at that site. Geologic conditions and rock mass characteristics can exhibit such a wide degree of variability that it is not possible to establish a single standardized approach. The scope of the program is determined by the level of complexity of the site geology, foundation loading characteristics, size, configuration, and structural performance of the bridge, acceptable levels of risk, experience of the agency, and other factors. Some of the information needed to establish the scope of site characterization may only be known following a preliminary study of the site.

Rock and IGM exhibit behaviors that are unique and require special techniques for application to engineering problems. Two aspects of rock behavior that are paramount are: (1) natural rock masses may exhibit a high degree of variability and (2) properties of a rock mass are determined by the combined properties of intact rock and naturally occurring discontinuities, such as joints, bedding planes, faults, and other structural features.

SITE GEOLOGY

Understanding the geologic environment provides information used to plan the more detailed, subsequent phases of exploration. Site geology refers to the physiography, surficial geology, and bedrock geology of the site. The starting point is a thorough survey of existing information. In many cases, existing data will enable identification of geologic features that will determine the feasibility of rock-socketed foundations or will have a major impact on their design or construction. The amount and quality of information gathered can then be used to establish the type and extent of additional data that will be required. General knowledge of the site geology is required in the first phase of the design process outlined in chapter one, *Conceptual Bridge Foundation Design*, to establish anticipated site conditions, feasibility of rock sockets, and conceptual evaluation of potential geotechnical hazards.

Sources of existing data include: geologic and topographic maps, publications, computer databases, aerial photographs, and consultation with other professionals. Many references are available that provide detailed information on sources and applications of existing data to geotechnical site characterization (e.g., Mayne et al. 2001). A detailed treatment of the topic is beyond the scope of this report and only the general aspects of such data sources will be summarized.

Geologic maps are used to transmit information about geologic features at or near the earth's surface. Maps are prepared at various scales and for a variety of purposes (Varnes 1974). A geologic map may be prepared to depict the general geology of a large region, for example bedrock geology of an entire state, or it may cover a relatively small area and contain detailed information about specific geologic features, for example engineering geology of a single quadrangle. A good starting point is the geologic map of the state. These maps are produced at a scale that makes it possible to identify the underlying bedrock formations in a general area. Often this is sufficient to know immediately whether a bridge is located where bedrock conditions are favorable or unfavorable for foundations in rock, or even whether bedrock exists at reasonable depth. Most state DOT geotechnical engineers and geologists with experience have familiarity with the geology of their state and incorporate this step unconsciously. The next logical step is to determine if more detailed geologic maps or reports are available for the particular area in which the bridge is located. Sources of such maps and publications include U.S. Geological Survey and state Geological Surveys, university libraries, and Soil Conservation Service. The use of Internet search engines has added a powerful tool for locating such information and most governmental geologic publications can now be identified and obtained on-line. Detailed geologic maps normally provide useful information on characteristics of bedrock and, in some cases surficial, geology relevant to foundation engineering. These maps provide descriptions of rocks in terms of lithology (rock type, mineralogy, and genesis), age, and structure (strike and dip of sedimentary rocks). In addition, major structural features are identified, such as faults, folds, and contacts between rock units (formations or members). Geologic maps prepared specifically for engineering purposes may include data on discontinuity patterns and characteristics, rock material strength, Rock Mass Ratings (RMRs), groundwater conditions, and depth to bedrock (Radbruch-Hall et al. 1987). Many will identify geologic hazards such as swelling soils or rock, landslides, corrosion potential, karst, abandoned mines, and other information of value. If engineering geologic maps are available they are an essential tool that should be used.

The most practical aerial photographs for geotechnical purposes are black and white photographs taken with stereo overlap and with panchromatic film, from heights of between 500 m and 3,000 m, at scales of about 1:10,000 to 1:30,000. The higher level photographs provide a resolution most useful for larger-scale features such as topography, geology, and landform analysis, whereas the lower-level photographs provide more detail on geologic structure. Landslides and debris flows, major faults, bedding planes, continuous joint sets, rock outcrops, and surface water are some of the features that can be identified and are relevant to the siting of bridge structures.

A potentially valuable source of existing data may be consultation with other geoprofessionals with design or construction experience in the same rock units. Geotechnical engineers, geologists, groundwater hydrologists, contractors, mining company personnel, well drillers, etc., may be able to provide geotechnical engineering reports from nearby projects, photographic documentation of excavations or other construction works, and unpublished reports or testing data. In addition, such individuals are often willing to share relevant experience. Bedian (2004) describes a case history in which experience at an adjacent site was used to develop a value engineer proposal for the design of rock-socketed foundations for a high rise building.

The geotechnical literature contains many useful papers describing design, construction, and/or load testing of rock-socketed drilled shafts in which the focus is on a particular type of rock or a specific formation. For example, Hassan and O'Neill (1997) present correlations for side resistance of shafts in the Eagle Ford Shale, a rock unit commonly encountered in north-central Texas, most notably in the Dallas area. Results of load tests on drilled shafts in mica schist of the Wissahickon Formation, commonly encountered in Philadelphia and other parts of eastern Pennsylvania, are given in Koutsoftas (1981) and Yang et al. (2004). Turner et al. (1993) and Abu-Hejleh et al. (2003) consider side resistance from load tests on shafts socketed into Pierre and Denver Formation shales. McVay et al. (1992) present a thorough study on the design of shafts in Florida limestone. Numerous other examples could be cited. Whenever such publications are available they should be used as a source of background information during the planning phase of any project where the same rock units are present. Results of load tests at different locations, but in the same rock unit, cannot be applied without judgment and site-specific considerations, but they do provide a framework for considering design issues and may provide insight on expected performance. Similarly, publications describing construction challenges in certain geologic environments and strategies for addressing them can be useful. Schwartz (1987) described construction problems and recommended solutions for rock-socketed piers in Piedmont formations in the Atlanta area. Brown (1990) identified problems involved in construction of drilled shafts in the karstic limestone of northern Alabama and suggests methods and approaches that have been successful for dealing with such challenges. A literature review often is all that is necessary to locate this type of useful information.

Where bedrock is exposed in surface outcrops or excavations, field mapping is an essential step to obtaining information about rock mass characteristics relevant to design and construction of foundations. A site visit is recommended for reconnaissance and field mapping following a review of existing information. A competent engineering geologist or geotechnical engineer can make and record observations and measurements on rock exposures that may complement, or in some cases exceed, the information obtained from borings and core sampling. Rock type, hardness, composition, degree of weathering, orientation and characteristics of discontinuities, and other features of a rock mass may be readily assessed in outcrops or road cuts. Guidance on detailed geologic mapping of rock for engineering purposes is given in Murphy (1985), *Rock Slopes . . .* (1989), and ASTM D4879 (*Annual Book . . .* 2000). Photography of the rock mass can aid engineers and contractors in evaluating potential problems associated with a particular rock unit. The major limitation lies in whether the surface exposure is representative of the rock mass at a depth corresponding to foundation support. When rock coring and surface mapping demonstrate that surface exposures are representative, the surface exposures should be exploited for information. Figure 4 shows a bridge site where mapping of rock exposures could provide much of the relevant data for design of foundations.

FIGURE 4 Bridge site with surface exposures of foundation rock.

FIELD INVESTIGATIONS

Field methods for characterization of rock include geophysical methods, rock core drilling, and in situ testing. These activities normally are carried out during the Preliminary Foundation Design phase of the design process as described in chapter one, and would be used to provide a description of subsurface conditions and a preliminary subsurface profile. The detailed results of field investigations, including detailed boring logs, in situ testing results, and interpretation, would be included in the final geotechnical report prepared during the Final Foundation Design phase of Figure 2.

Geophysical Methods

Geophysical methods, in conjunction with borings, can provide useful information in areas underlain by rock. The most common application of geophysics is to determine depth to bedrock. When correlated with data from borings, geophysical methods provide depth to bedrock information over a large area, eliminating some of the uncertainty associated with interpolations of bedrock depths for locations between borings.

Geophysical methods are based on measuring the transmission of electromagnetic or mechanical waves through the ground. Signal transmission is affected by differences in the physical properties of geomaterials. By transmitting electromagnetic or seismic signals and measuring their arrival at other locations, changes in material properties can be located. In some cases, the material properties can also be quantified. For foundation site characterization, geophysical methods can be placed into two general categories, those conducted from the ground surface (noninvasive) and those conducted in boreholes (invasive). When grouped according to method, the six major categories are: seismic, electromagnetic, electrical, magnetic, radar, and gravity. Basic descriptions of geophysical methods and their application to geotechnical engineering are given by the U.S. Army Corps of Engineers ("Geophysical Exploration . . . ." 1995) and Mayne et al. (2001).

NCHRP Synthesis 357: Use of Geophysics for Transportation Projects (Sirles 2006) provides a comprehensive overview of the topic and additional survey data relevant to this study. Table 1 identifies the primary and secondary methods used to investigate selected subsurface objectives. The table is an abridged version from the Sirles report (2006) in which only objectives pertaining to foundation investigations are included. The survey of transportation agencies for this project identified "seismic" as the most widely used geophysical methods and "mapping rock" as the most widely used application of geophysics. Mapping karst or other voids was also identified as a major objective.

Results of the survey for this study are consistent with those of Sirles (2006). The most frequently applied method is seismic refraction, which is based on measuring the travel time of compressional waves through the subsurface. Upon striking a boundary between two media of different properties the direction of travel is changed (refraction). This change in direction is used to deduce the subsurface profile. Figure 5*a* illustrates the basic idea for a simple two-layer profile in which soil of lower seismic velocity (V_{p1}) overlies rock of higher seismic velocity (V_{p2}). A plot of distance from the source versus travel time (Figure 5*b*) exhibits a clear change in slope corresponding to the depth of the interface. The equipment consists of a shock wave source (typically a hammer striking a steel plate), a series of geophones to measure seismic wave arrival, and a seismograph with oscilloscope. The seismograph records the impact and geophone signals in a timed sequence and stores the data digitally. The technique is rapid, accurate, and relatively economical when applied correctly. The interpretation theory is relatively straightforward and equipment is readily available. The most significant limitations are that it is incapable of detecting material of lower velocity (lower density) underlying higher velocity (higher density) and that thin layers sometimes are not detectable. For these reasons, it is important not to rely exclusively on seismic refraction, but to verify depth to rock in several borings and correlate the seismic refraction signals to the boring results. Seismic velocity, as determined from seismic refraction measurements, can be correlated to small-strain dynamic modulus of soil and rock by the following relationships:

$$E_d = 2(1+\nu_d)\rho V_s^2 \tag{1}$$

$$E_d = \frac{(1-2\nu)(1+\nu_d)}{(1-\nu_d)}\rho V_p^2 \tag{2}$$

in which E_d = small-strain dynamic modulus, ν_d = small-strain dynamic Poisson's ratio, ρ = mass density, V_s = shear wave velocity, and V_p = compressional wave velocity. Eqs. 1 and 2 are based on the assumption that the rock mass is a homogeneous, isotropic, elastic solid. Because most rock masses depart significantly from this assumption, elastic modulus values calculated from seismic wave velocities are normally larger than values measured in static field load

TABLE 1
GEOPHYSICAL METHODS AND APPLICATIONS (after Sirles 2006)

Methods	Techniques													
	Seismic					Electromagnetic				Electrical		Other		
Investigation Objectives	Seismic Refraction	Seismic Reflection	Seismic Tomography	Shear Wave	Surface Wave (SASW, MASW, & Passive)	EM31—Terrain Conductivity	EM34—Terrain Conductivity	EM61—Time–Domain Metal Detector	Time–Domain EM Soundings	Electrical Resistivity/P	Electrical Resistivity Tomography/P	Gravity	Magnetics	Ground Penetrating Radar
Bedrock Depth	P	P	P		P				S	S		S		
Rippability	P		P	P										
Lateral & Vertical Variation in Rock or Soil Strength	P		P	P	P									
Location of Faults and Fracture Zones	P	P	P	P	S	S	S		S	S	S	S		
Karst Features			P		P	S	S				P			P

Notes: P = primary; S = secondary; blank = techniques should not be used; EM = electromagnetic; SASW = spectral analysis of surface waves; MASW = multi-channel analysis of surface waves.

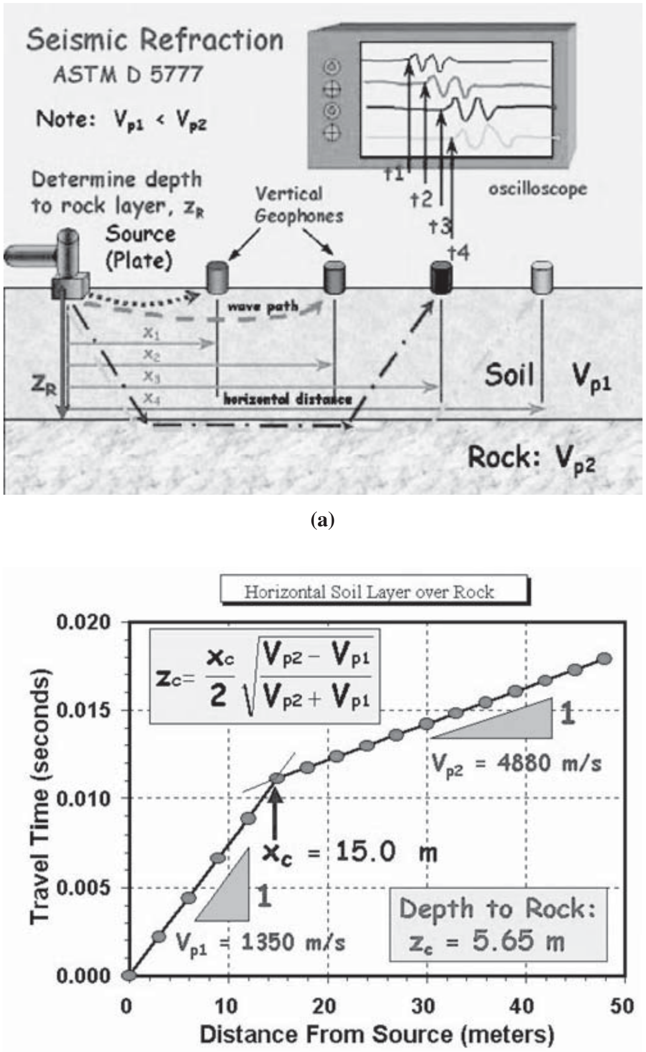
tests, such as plate bearing or pressure chamber tests. Alternatively, a method that correlates rock mass modulus to shear wave frequency has been shown to provide a reasonable first-order estimate of modulus. Figure 6 shows the relationship between in situ modulus and shear wave frequency using a hammer seismograph, as described by Bieniawski (1978). The data can be fit to a straight line by

$$E_M = 0.054f - 9.2 \tag{3}$$

where E_M = rock mass static modulus (GPa) and f = shear wave frequency (hertz) from the hammer blow received at distances of up to 30 m on a rock surface.

Resistivity is a fundamental electrical property of geomaterials that varies with material type and water content. To measure resistivity from the ground surface (Figure 7), electrical current is induced through two current electrodes (C_1 and C_2), while change in voltage is measured by two potential electrodes (P_1 and P_2). Apparent electrical resistivity is then calculated as a function of the measured voltage difference, the induced current, and spacing between electrodes. Two techniques are used. In a sounding survey, the centerline of the electrodes is fixed while the spacing of the electrodes is increased for successive measurements. The depth of material subjected to current increases with increasing electrode spacing. Therefore, changes in measured apparent resistivity with increasing electrode spacing are indicative of a change in material at depth. In this way, variations in material properties with depth (layering) can be determined. The second method is a profiling survey in which the electrode spacing is fixed but the electrode group is moved horizontally along a line (profile) between measurements. Changes in measured apparent resistivity are used to deduce lateral variations in material type. Electrical resistivity methods are inexpensive and best used to complement seismic refraction surveys and borings. The technique has advantages for identifying soft materials in between borings. Limitations are that lateral changes in apparent resistivity can be interpreted incorrectly as depth related. For this and other reasons, depth determinations can be in error, which is why it is important to use resistivity surveys in conjunction with other methods.

The use of multi-electrode resistivity arrays shows promise for detecting detailed subsurface profiles in karst terranes, one of the most difficult geologic environments for rock-socketed foundations. Dunscomb and Rehwoldt (1999) showed that two-dimensional (2-D) profiling using multi-electrode arrays provides reasonable resolution for imaging features such as pinnacled bedrock surfaces, overhanging rock ledges, fracture zones, and voids within the rock mass and in the soil overburden. Hiltunen and Roth (2004) present the results of multiple-electrode resistivity surveys at two bridge sites on I-99 in Pennsylvania. The resistivity profiles were compared with data from geotechnical borings. Both sites are located in karst underlain by either dolomite or limestone. The resistivity profiles provided a very good match to the



(a)

(b)

FIGURE 5 Seismic refraction method (Mayne et al. 2001): (*a*) field setup and procedures; (*b*) data reduction for depth to hard layer.

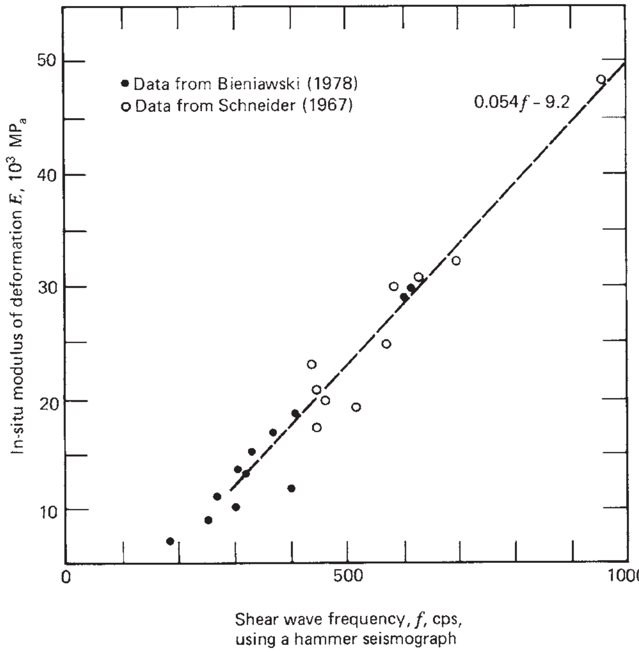


FIGURE 6 Rock mass modulus versus shear wave frequency by Bieniawski (Goodman 1980).

stratigraphy observed in borings, particularly for top-of-rock profile. Figure 8 shows a resistivity tomogram at one of the bridge pier sites, in which the top-of-rock profile is well-defined by the dark layer. Inclusions of rock in the overlying soil are also clearly defined. This technology should be considered for any site where a rock surface profile is required and would provide valuable information for both design and construction of rock-socketed foundations. Table 1 identifies electrical resistivity tomography profiling as a primary method for investigating karstic conditions and as a secondary method for measuring depth to bedrock.

Other geophysical methods have potential for rock sites, but have yet to be exploited specifically for applications to foundations in rock. These include downhole and crosshole seismic methods. Downhole seismic *p*-wave is based on measuring arrival times in boreholes of seismic waves generated at the ground surface. Crosshole seismic involves measuring travel times of seismic waves between boreholes. Both methods provide depth to rock, and *s*-wave velocities, dynamic shear modulus, small-strain Young's modulus, and Poisson's ratio. Crosshole tomography is based on computer analysis of crosshole seismic or resistivity data to produce a 3-dimensional (3-D) representation of subsurface conditions. These techniques are more expensive and require specialized expertise for data interpretation, but may be cost-effective for large structures where the detailed information enables a more cost-effective design or eliminates uncertainty that may otherwise lead to construction cost overruns.

All geophysical methods have limitations associated with the underlying physics, the equipment, and the individuals running the test and providing interpretation of the data. The study by Sirles (2006) includes several informative case histories from state DOTs of both successful and unsuccessful projects. The single case history related to a bridge foundation investigation is one of a failure to provide accurate

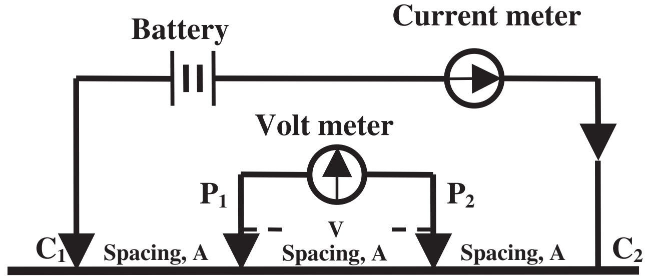


FIGURE 7 Field configuration for resistivity test.

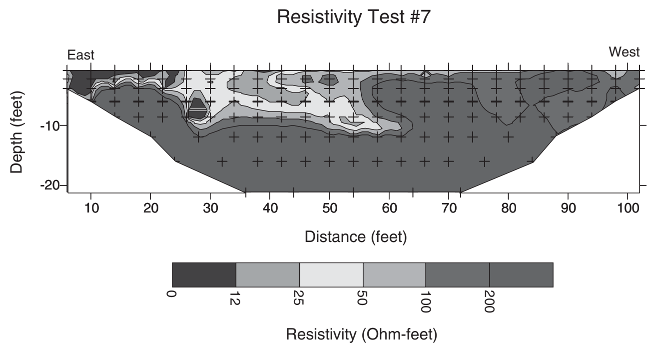


FIGURE 8 Resistivity tomogram at Pennsylvania bridge site in karst (Hiltunen and Roth 2004).

depths to bedrock in a river channel using both seismic refraction and an electrical resistivity sounding survey. Reasons cited for the failure include loss of geophones owing to running water and ice, instrumentation malfunctions, excessive background noise, differences of opinion between consultants on data interpretation, and discrepancies between top of rock from geophysical results and borings. Although this is not believed to be a typical case, it demonstrates some real world lessons.

Additional findings by Sirles (2006) are that "in-house geoscientists and engineers do not understand the value, the benefit, or the science of geophysics for their projects." However, several factors point to geophysics becoming more widely accepted and implemented as a tool in the transportation industry. These include a manual published by FHWA and available on-line (http://www.cflhd.gov/geotechnical), additional programs aimed at training of agency personnel, and increasing levels of experience.

Borings

Borings provide the most direct evidence of subsurface conditions at a specific site. They furnish detailed information on stratigraphy and samples of soil and rock from which engineering properties are determined. Borings also provide the means for conducting in situ tests, installation of instrumentation, and observing groundwater conditions. Conventional soil boring and testing equipment is used to drill through overlying soil deposits and to determine depth to bedrock. Once encountered, the most widely used technique for investigating rock for the purpose of foundation design is core drilling. Samples are obtained for rock classification and determining rock properties important to both design and construction. A core sample can be examined physically and tested, providing information that is hard to obtain by any other methods.

Rock core drilling is accomplished using rotary drill equipment, usually the same truck- or skid-mounted rigs used for soil drilling and sampling. A hollow coring tube equipped with a diamond or tungsten–carbide cutting bit is rotated and forced downward to form an annular ring while preserving a central rock core. Standard core barrel lengths are 1.5 m and 3 m (5 ft and 10 ft). Fluid, usually water but possibly drilling mud, is circulated for cooling at the cutting interface and removal of cuttings. Selecting the proper tools and equipment to match the conditions and the expertise of an experienced drill crew are essential elements of a successful core drilling operation. Once rock is encountered, coring normally is continuous to the bottom of the hole. Where the rock being sampled is deep, wire line drilling, in which the core barrel is retrieved through the drill stem, eliminates the need to remove and reinsert the entire drill stem and can save considerable time. If sampling is not continuous, drilling in between core samples can be accomplished using solid bits.

Rock coring bits and barrels are available in standardized sizes and notations. Important considerations in core barrel selection are: (1) core recovery and (2) the ability to determine the orientation of rock mass structural features relative to the core. Core recovery is most important in highly fractured and weak rock layers, because these zones are typically critical for evaluation of foundation–rock load transfer. For sampling of competent rock, bits and core barrels that provide a minimum of 50-mm-diameter (nominal) core are adequate for providing samples required for index tests, rock quality designation (RQD), laboratory specimens for strength testing, and evaluating the conditions of discontinuities. For example, NWM (formerly NX) diamond bit and rock core equipment drills a 76-mm (3-in.) diameter hole and provides a 54-mm (2.125-in.) diameter rock core. When weak, soft, or highly fractured rock is present, it may be necessary to use larger diameter bits and core barrels to improve core recovery and to obtain samples from which laboratory strength specimens can be prepared. Coring tools up to 150 mm (6 in.) in diameter are used. A highly recommended practice for best core recovery is to use triple-tube core barrels. The inner sampling tube does not rotate during drilling and is removed by pushing instead of hammering; features that minimize disturbance. Thorough descriptions of coring equipment and techniques are given in Acker (1974), AASHTO *Manual on Subsurface Investigations* (1988), Mayne et al. (2001), and U.S. Army Corps of Engineers ("Geotechnical Investigations" 2001).

Steeply dipping or near-vertical bedding or jointing may go undetected in holes drilled vertically (Terzaghi 1965). Such features can significantly influence the strength and deformability of rock foundations. Inclined (nonvertical) drilling provides the opportunity to detect the orientation and characteristics of near-vertical features. Oriented core refers to any method that provides a way to determine the geometrical orientation of planar structural features, such as bedding, joints, fractures, etc., with respect to the geometrical orientation of the core. One approach is to mark the core with a special engraving tool so that the orientation of the discontinuity relative to the core is preserved and the orientation of the discontinuity (strike and dip) can be determined accurately (Goodman 1976). A method used with wire line drilling involves making an impression of the core in clay. The combination of inclined and oriented coring techniques can provide an effective tool for characterizing orientation of discontinuities in complexly fractured rock masses. Rock core orienting methods are covered in more detail in the AASHTO *Manual on Subsurface Investigations* (1988) and are also reviewed and compared with borehole televiewer methods by Eliassen et al. (2005).

Depth and Spacing of Boreholes

O'Neill and Reese (1999) recommend the number of borings to be made per drilled shaft location at bridge sites when the material to be excavated is unclassified (Table 2). Unclassified means the contractor is paid by the unit of excavation depth (meters or feet) regardless of the material encountered. For rock sites, these recommendations should be considered a minimum. If possible, it is recommended to locate one boring at every rock-socketed shaft. In practice, this is not always possible and factors such as experience, site access, degree of subsurface variability, geology, and importance of the structure will be considered. If materials are classified for payment purposes, it becomes more important to locate a boring at every drilled shaft location for the purpose of making accurate cost estimates and for contractors to base their bids on knowledge of the materials to be excavated. Where subsurface conditions exhibit extreme variations over short distances, multiple borings at each shaft location can reduce the risk of founding a shaft on soil instead of rock. For example, large-diameter, nonredundant shafts in karstic limestone may require multiple borings at each shaft location to determine that the entire base will be founded in rock and to identify voids or zones of soil beneath the base that may affect load-settlement behavior of each shaft.

The draft 2006 Interim AASHTO *LRFD Bridge Design Specifications* recommends the following for depth of borings below anticipated tip elevations:

TABLE 2
RECOMMENDED FREQUENCY OF BORINGS, DRILLED SHAFT FOUNDATIONS FOR BRIDGES, UNCLASSIFIED EXCAVATION

Redundancy Condition	Shaft Diameter (m)	Guideline
Single-column, single shaft foundations	All	One boring per shaft
Redundant, multiple-shaft foundations	≥1.8 m (6 ft)	One boring per shaft
Redundant, multiple-shaft foundations	1.2–1.8 m (4–6 ft)	One boring per two shafts
Redundant, multiple-shaft foundations	<1.2 m (4 ft)	One boring per four shafts

Source: ONeill and Reese 1999.

> For shafts supported on or extending into rock, a minimum of 3 m of rock core, or a length of rock core equal to at least three times the shaft diameter for isolated shafts or two times the maximum shaft group dimension, whichever is greater, shall be extended below the anticipated shaft tip elevation to determine the physical characteristics of rock within the zone of foundation influence.

If the tip elevation changes at some point during the project, additional drilling may be required to meet this recommendation. O'Neill and Reese (1999) provide the following guidance on boring depth. When the RQD is less than 50%, extend boring depths to at least 125% of the expected depths of the drilled shaft bases plus two base diameters. If RQD values are greater than approximately 50% at the planned base elevation, borings only need be extended to the expected base elevation plus two base diameters as long as the RQD remains above 50%. The rationale is that it is not likely the shafts will need to be deepened once the actual strata are exposed. This approach requires that foundation diameters and depths be estimated before the boring program and that RQD be determined during drilling. The approach described is only a general suggestion and local geologic conditions may dictate other criteria for boring depths. If in the course of design or construction it becomes necessary to deepen the shafts, supplementary borings should be taken.

An available, but not widely used tool for subsurface investigation is to drill one or more large-diameter borings or to have a drilled shaft contractor install a full-sized test excavation. Large-diameter borings can be made with augers in soft rock and with core barrels in hard rock. The sidewalls of the boring or shaft can be examined directly (with appropriate safety measures) or with downhole cameras. Observations can then be made of rock mass features, including degree of roughness and general quality of the drilled surfaces, and fracture patterns. Large-diameter holes provide access for obtaining high-quality undisturbed samples and may be used for performing in situ plate load tests to measure rock mass modulus. If a full-size excavation is made by a drilled shaft contractor, information of value to both engineers and contractors is obtained. In Figure 3, "constructability" is one of the items to be determined during the site characterization. A full-sized excavation is the most direct method for obtaining this information.

Downhole devices are available for borehole viewing and photography, including borescopes, photographic cameras, and television cameras. A visual image of rock in the sidewalls of a boring provides information on structural features that may add significantly to the overall picture of subsurface geology. Advantages and disadvantages of some remote viewing devices are discussed in "Geotechnical Investigations" (2001); however, the technologies for borehole imaging are advancing rapidly and the user should consult commercial providers for the most up-to-date information. These devices are effective for examining soft zones for which core may not have been recovered, determination of dip and strike of important structural features, and viewing of cavities such as solution voids, open joints, and lava tunnels in volcanic rocks.

Borehole televiewers provide high-resolution images showing rock mass structural and textural features and accurate measurement of dip and dip direction of structural features without the use of oriented core. Optical televiewers (OTV) generate a high-resolution digital color image of the inside of the borehole wall and are capable of resolving fractures as narrow as 0.1 mm with a radial resolution of 1 degree (Eliassen et al. 2005). The OTV can be operated in air- or fluid-filled boreholes; however, fluid requires thorough flushing before image acquisition is undertaken. Acoustic televiewers (ATV) produce images of the borehole wall based on the amplitude and travel time of acoustic signals reflected from the borehole wall. A portion of the reflected energy is lost in voids or fractures, producing dark bands on the amplitude log. Travel time measurements allow reconstruction of the borehole shape, making it possible to generate a 3-D representation of a borehole.

Both types of televiewers orient their image data using a three-component fluxgate magnetometer and a three-component tilt meter incorporated into the tool. Before interpretation, the image is rotated to a common reference direction, either magnetic north or the high side of the borehole. Planar features that intersect the borehole wall produce sinusoidal traces in the "unwrapped," or 2-D, televiewer image. Using the reference direction recorded during logging, sinusoids can be analyzed to produce dip and dip directions of structural features. Figure 9 shows OTV and ATV images of the same borehole and illustrates some advantages of each device. The OTV is able to provide a color image of the dike and excellent imaging of the texture of the granite. The ATV highlights fracturing within the diorite. The California DOT (Caltrans) reports using the ATV to provide very-high-resolution sonic images in the format of a 3-D "pseudo-core," as illustrated in Figure 10.

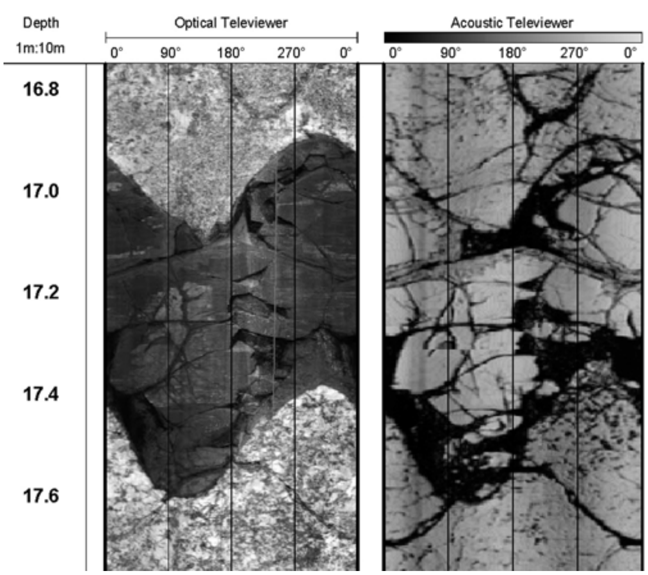


FIGURE 9 Optical and acoustic televiewer images of a 50-cm diorite dike in granite (Eliassen et al. 2005).

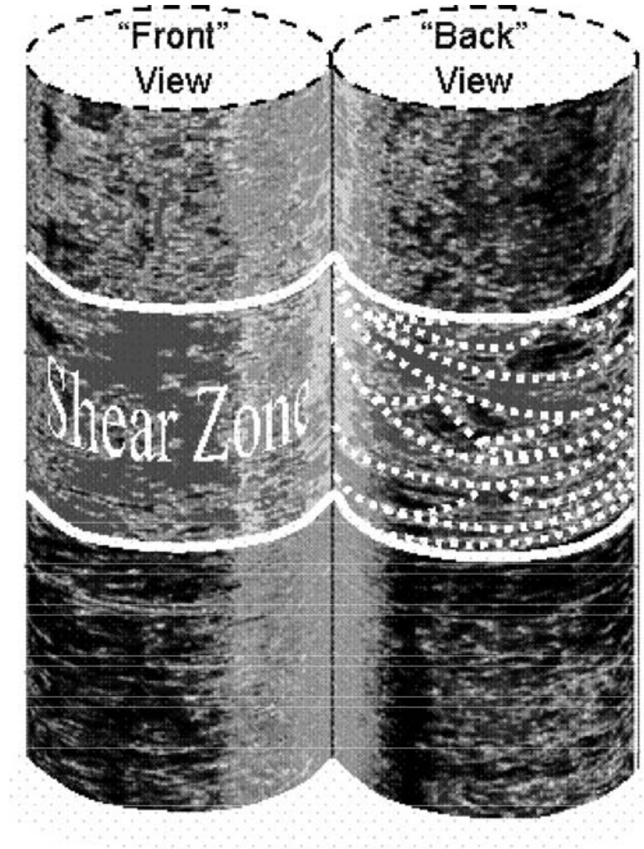


FIGURE 10 An acoustic television log (Caltrans 2005).

According to Eliassen et al. (2005), use of optical and acoustic televiewer equipment is gaining popularity over oriented coring techniques because it is generally less labor intensive and is particularly useful where access or ability to drill inclined holes is limited or where local drilling companies lack the equipment necessary to collect oriented cores. However, to date, this technology is being applied to site characterization for rock slope engineering and underground openings, and is not being used in foundation investigations. Eliassen et al. (2005) note further that televiewer logs are best used to supplement data obtained from quality rock coring, which provides samples for laboratory testing, assessment of joint and discontinuity planes, and correlation of lithologic and geologic boundaries with geophysical data. The authors suggest that drilling time and costs can be optimized with appropriate combinations of coring and less expensive air rotary boreholes logged with OTV and ATV equipment. Borehole televiewing may be most useful in rock-socket applications at sites where the structural orientation of discontinuities is a significant factor in foundation stability. For example, some modes of bearing capacity failure (described in chapter three) depend on the orientation of discontinuities in the rock mass below the socket base. LaFronz et al. (2003) describe use of OTV as part of the subsurface investigation for the Colorado River Bridge at Hoover Dam. The primary purpose was to obtain structural data to develop recommendations for excavation of cut slopes at the abutment foundations.

GEOLOGIC AND INDEX PROPERTIES OF ROCK

The most basic characterization of rock for engineering purposes is a description of rock core based on visual and physical examination. The International Society of Rock Mechanics (ISRM) proposed a standardized method for descriptions of rock masses from mapping and core logging ("Basic Geotechnical Description of Rock Masses" 1981). A summary of the ISRM method as given by Wyllie (1999) is adopted in the FHWA manuals on subsurface investigations and soil and rock properties (Mayne et al. 2001; Sabatini et al. 2002) and is summarized here.

A rock mass is described in terms of five categories of properties, as follows:

1. Rock Material Description—a. Rock type, b. Wall strength, c. Weathering
2. Discontinuity Description—d. Type, e. Orientation, f. Roughness, g. Aperture
3. Infilling—h. Infilling type and width
4. Rock Mass Description—i. Spacing, j. Persistence, k. Number of sets, l. Block size/shape
5. Groundwater—m. Seepage.

Each of the 13 parameters listed (a through m) is assigned a description using standardized terminology. Descriptive terms are given in Tables 3 through 6 and in Figure 11, which is an example of a Key used for entering rock descriptions on a coring log and includes details of several categories.

Rock Material Descriptors

Rock type is defined in terms of origin (igneous, sedimentary, or metamorphic) and then further classified into one of the rock types listed in Table 3 based on lithologic characteristics that include color, fabric (microstructural and textural features), grain size and shape (Tables 4 and 5), and mineralogy. Sedimentary rock descriptions should include bedding thickness (Table 6). The rock unit name, which may be a formal name of a formation or an informal local name, should be identified; for example, Bearpaw Shale or Sherman Granite.

Compressive strength of rock core can be evaluated using simple field tests with equipment commonly available (knife, rock hammer, etc.) and summarized in the Key of Figure 11 ("Rock Strength") or evaluated from point load

TABLE 3
ROCK GROUPS AND TYPES

	Igneous	
Intrusive (coarse-grained)	Extrusive (fine-grained)	Pyroclastic
Granite	Rhyolite	Obsidian
Syenite	Trachyte	Pumice
Diorite	Andesite	Tuff
Diabase	Basalt	
Gabbro		
Peridotite		
Pegmatite		
	Sedimentary	
Clastic (sediment)	(chemically formed)	(organic remains)
Shale	Limestone	Chalk
Mudstone	Dolomite	Coquina
Claystone	Gypsum	Lignite
Siltstone	Halite	Coal
Conglomerate		
Limestone, oolitic		
	Metamorphic	
Foliated	Nonfoliated	
Slate	Quartzite	
Phyllite	Amphibolite	
Schist	Marble	
Gneiss	Hornfels	

TABLE 4
TERMS TO DESCRIBE GRAIN SIZE OF SEDIMENTARY ROCK

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

TABLE 5
TERMS TO DESCRIBE GRAIN SHAPE (for sedimentary rocks)

Description	Characteristic
Angular	Showing very little evidence of wear. Grain edges and corners are sharp. Secondary corners are numerous and sharp.
Subangular	Showing definite effects of wear. Grain edges and corners are slightly rounded off. Secondary corners are slightly less numerous and slightly less sharp than in angular grains.
Subrounded	Showing considerable wear. Grain edges and corners are rounded to smooth curves. Secondary corners are reduced greatly in number and highly rounded.
Rounded	Showing extreme wear. Grain edges and corners are smoothed off to broad curves. Secondary corners are few in number and rounded.
Well-rounded	Completely worn. Grain edges and corners are not present. No secondary edges or corners are present.

TABLE 6
TERMS TO DESCRIBE STRATUM THICKNESS

Descriptive Term	Stratum Thickness
Very thickly bedded	>1 m
Thickly bedded	0.5 to 1.0 m
Thinly bedded	50 mm to 500 mm
Very thinly bedded	10 mm to 50 mm
Laminated	2.5 mm to 10 mm
Thinly laminated	<2.5 mm

tests or uniaxial compression tests conducted on specimens. The rock strength descriptions given at the bottom of the second page of the Key correspond to the seven categories of rock strength, R0 through R6, of the ISRM ("Basic Geotechnical Description of Rock Masses" 1981), with R0 corresponding to extremely weak rock and R6 corresponding to extremely strong rock. The degree of physical disintegration or chemical alteration of rock can be described by the terms and abbreviations given in the Key. Weathering and alteration reduces shear strength of both intact rock and discontinuities.

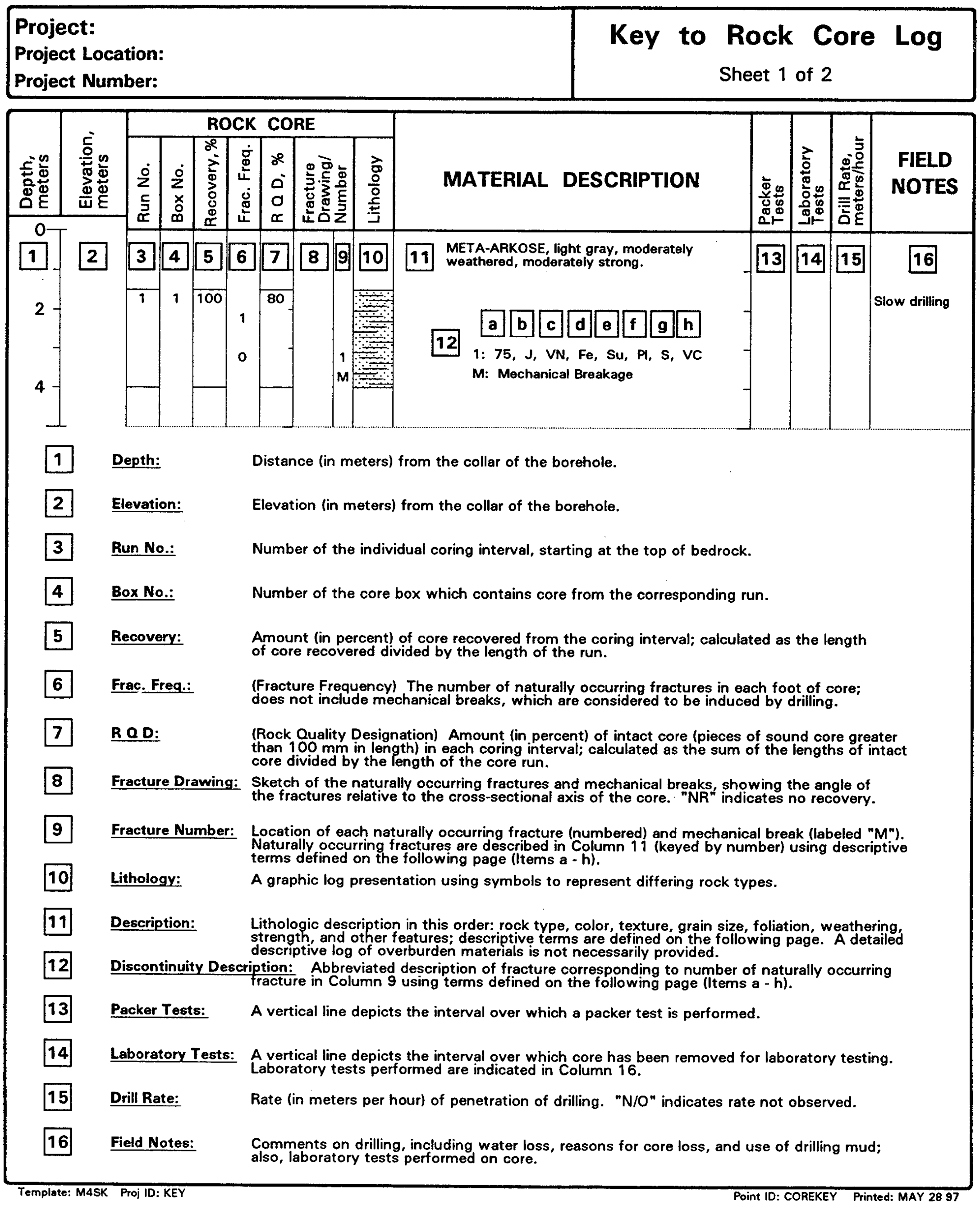


FIGURE 11 Key for rock core description (sheet 1).

Project: Project Location: Project Number:	**Key to Rock Core Log** Sheet 2 of 2

Depth, meters	Elevation, meters	ROCK CORE							MATERIAL DESCRIPTION	Packer Tests	Laboratory Tests	Drill Rate, meters/hour	FIELD NOTES
		Run No.	Box No.	Recovery, %	Frac. Freq.	R Q D, %	Fracture Drawing/ Number	Lithology					

KEY TO DESCRIPTIVE TERMS USED ON CORE LOGS

DISCONTINUITY DESCRIPTORS

a Dip of fracture surface measured relative to horizontal

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

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

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

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

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

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

h Discontinuity Spacing (meters):
- EW - Extremely Wide (>20)
- W - Wide (7-20)
- M - Moderate (2.5-7)
- C - Close (0.7-2.5)
- VC - Very Close (<0.7)

ROCK WEATHERING / ALTERATION

Description	Recognition
Residual Soil	Original minerals of rock have been entirely decomposed to secondary minerals, and original rock fabric is not apparent; material can be easily broken by hand
Completely Weathered/Altered	Original minerals of rock have been almost entirely decomposed to secondary minerals, minerals, although original fabric may be intact; material can be granulated by hand
Highly Weathered/Altered	More than half of the rock is decomposed; rock is weakened so that a minimum 50-mm-diameter sample can be broken readily by hand across rock fabric
Moderately Weathered/Altered	Rock is discolored and noticeably weakened, but less than half is decomposed; a minimum 50-mm-diameter sample cannot be broken readily by hand across rock fabric
Slightly Weathered/Altered	Rock is slightly discolored, but not noticeably lower in strength than fresh rock
Fresh	Rock shows no discoloration, loss of strength, or other effect of weathering/alteration

ROCK STRENGTH

Description	Recognition	Approximate Uniaxial Compressive Strength (kPa)
Extremely Weak Rock	Can be indented by thumbnail	250 - 1,000
Very Weak Rock	Can be peeled by pocket knife	1,000 - 5,000
Weak Rock	Can be peeled with difficulty by pocket knife	5,000 - 25,000
Medium Strong Rock	Can be indented 5 mm with sharp end of pick	25,000 - 50,000
Strong Rock	Requires one hammer blow to fracture	50,000 - 100,000
Very Strong Rock	Requires many hammer blows to fracture	100,000 - 250,000
Extremely Strong Rock	Can only be chipped with hammer blows	> 250,000

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FIGURE 11 (*continued*) (sheet 2).

Discontinuity Descriptors

A discontinuity is defined as any surface across which any mechanical property of a rock mass is discontinuous. Discontinuity descriptors are summarized in Figure 11 (Key), items a through g. Types of discontinuities include faults, joints, shear planes, foliation, veins, and bedding. Orientation refers to the measured dip and dip direction of the surface (or dip and strike). Dip is defined as the maximum angle of the plane to the horizontal and dip direction (strike) is the direction of the horizontal trace of the line of dip measured clockwise from north, in degrees. Determination of dip and dip direction from core samples is possible using oriented coring techniques, borehole televiewers, downhole cameras, or other devices capable of establishing orientation of the discontinuity relative to the core. Roughness and surface shape of joint surfaces is best measured in the field on exposed surfaces at least 2 m in length and can be described using the terms in the Key or quantified in terms of a Joint Roughness Coefficient (Barton 1973). Aperture is the width of a discontinuity with no infilling and can be classified according to Box c of the Key.

Infilling

Infilling is the term for material separating adjacent rock walls of discontinuities. Infilling is described in terms of its type, amount, and width (Key). Additional laboratory testing may be conducted to determine soil classification and shear strength of infilling materials. Direct shear tests provide a means to measure shear strength of joints with infilling, as described by Wyllie and Norrish (1996). Infilling properties vary widely and can have a significant influence on rock mass strength (RMS), compressibility, and permeability.

Rock Mass Descriptors

Spacing is the perpendicular distance between adjacent discontinuities. Spacing has a major influence on seepage and mechanical behavior and can be described using the terms in Figure 11 (Key). Persistence refers to the continuous length or area of a discontinuity and requires field exposures for its determination.

The number of sets of intersecting discontinuities has a major effect on RMS and compressibility. As the number of sets increases, the extent to which the rock mass can deform without failure of intact rock also increases. Field mapping or observations made in exploratory pits or large excavations provide the best opportunity to map multiple sets of discontinuities. Block size and shape is determined by spacing, persistence, and number of intersecting sets of discontinuities. Descriptive terms include blocky, tabular, shattered, and columnar, while size ranges from small (<0.0002 m^3) to very large (>8 m^3).

Seepage

Field observations of seepage from discontinuities should be described whenever it can be observed. The presence and type of infilling controls joint permeability and should be described wherever seepage is observed. Seepage can range from dry to continuous flow under high pore water pressure

Rock Quality Designation

A simple and widely used measure of rock mass quality is provided by the RQD (rock quality designation, ASTM D6032). RQD is equal to the sum of the lengths of sound pieces of core recovered, greater than 100 mm (4 in.) in length, expressed as a percentage of the length of the core run. Originally introduced by Deere (1964), the RQD was evaluated by Deere and Deere (1989), who recommended modifications to the original procedure after evaluating its field use. Figure 12 illustrates the recommended procedure. Several factors must be evaluated properly for RQD to provide reliable results.

RQD was originally recommended for NX size core, but can also be used with the somewhat smaller NQ wireline sizes and with larger wire line sizes and other core sizes up to 150 mm (6 in.). RQD based on the smaller BQ and BX cores or with single-tube core barrels is discouraged because of core breakage. Core segment lengths should be measured along the centerline or axis of the core, as shown in Figure 12.

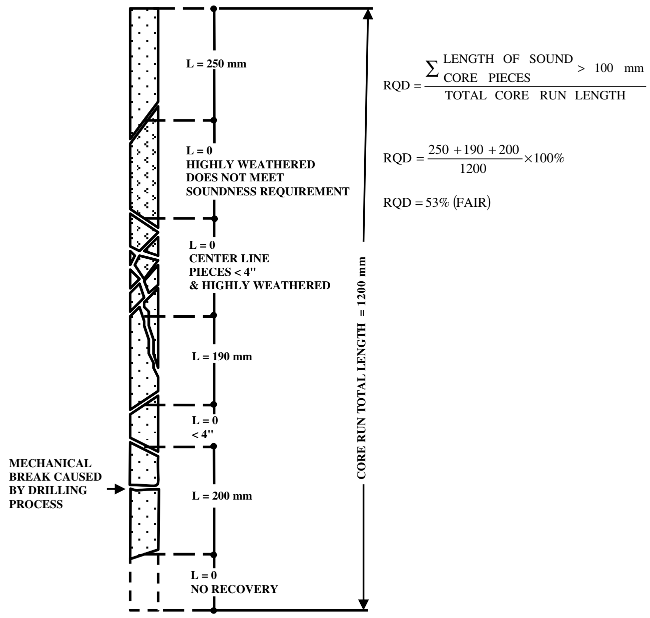


FIGURE 12 RQD determination of rock core (after Deere and Deere 1989).

Only natural fractures such as joints or shear planes should be considered when calculating RQD. Core breaks caused by drilling or handling should be fitted together and the pieces counted as intact lengths. Drilling breaks may be identified by fresh surfaces. For some laminated rocks it may be difficult to distinguish natural fractures from those caused by drilling. For characterization of rock mass behavior relevant to foundation design it is conservative to not count the length near horizontal breaks. RQD should be performed as soon as possible after the core is retrieved to avoid the effects of deterioration, which may include slaking and separation of core along bedding planes, especially in moisture-sensitive rocks like some shales. It is also desirable because RQD is a quantitative measure of core quality at the time of drilling when the rock core is "fresh" and most representative of in situ conditions.

Rock assigned a weathering classification of "highly weathered" or above should not be included in the determination of RQD. RQD measurements assume that core recovery is at or near 100%. As core recovery varies from 100%, explanatory notes may be required to describe the reason for the variation and the effect on RQD. In some cases, RQD will have to be determined on the basis of total length of core recovered, rather than on the length of rock cored. One state (Florida) uses percent core recovery as an index of rock quality in limestone.

A general description of rock mass quality based on RQD is given here. Its wide use and ease of measurement make it an important piece of information to be gathered on all core holes. Taken alone, RQD should be considered only as an approximate measure of overall rock quality. RQD is most useful when combined with other parameters accounting for rock strength, deformability, and discontinuity characteristics. As discussed in subsequent sections of this report, many of the rock mass classification systems in use today incorporate RQD as a key parameter.

Rock Mass Description	RQD
Excellent	90–100
Good	75–90
Fair	50–75
Poor	25–50
Very Poor	<25

ENGINEERING PROPERTIES OF ROCK

Laboratory Tests on Intact Rock

Intact rock refers to the consolidated and cemented assemblage of mineral particles forming the rock material, excluding the effects of macro-scale discontinuities such as joints, bedding planes, minor faults, or other recurrent planar fractures. The term rock mass is used to describe the system comprised of intact rock and discontinuities. The characteristics of intact rock are determined from hand specimens or rock core. Properties of intact rock required for proper characterization of the rock mass and that are relevant to foundation design include strength and deformability. For some rock types, the potential for degradation on exposure to atmospheric conditions may also need to be evaluated. Some design methods incorporate properties of intact rock directly; for example, correlations between ultimate unit side resistance and uniaxial compressive strength. However, most analytical treatments of foundation capacity and load-deformation response incorporate the strength and deformability of intact rock into rock mass models that also account for the effects of discontinuities, rock quality, and other factors.

Table 7 lists the laboratory tests for intact rock most commonly done for foundation design and gives the ASTM Standard Designation for each test. More thorough coverage of laboratory testing of intact rock is given by Mayne et al. (2001), the *Rock Testing Handbook* (1993), and the AASHTO *Manual on Subsurface Investigations* (1988).

Engineering properties of intact rock that are used most often for foundation design are uniaxial compressive strength (q_u) and elastic modulus (E_R). The compressive strength of intact rock is determined by applying a vertical compressive force to an unconfined cylindrical specimen prepared from rock core. The peak load is divided by the cross-sectional area of the specimen to obtain the uniaxial compressive strength (q_u). The ASTM procedure (D2938) specifies tolerances on smoothness over the specimen length, flatness of the ends, the degree to which specimen ends are perpendicular to the length, and length-to-diameter ratio. Uniaxial compressive strength of intact rock is used in empirical correlations to evaluate ultimate side and base resistances under axial loading; ultimate limit pressure under lateral loading; and, by contractors, to assess constructability.

Elastic modulus of intact rock is measured during conduct of the uniaxial compression test by measuring deformation as a function of load. It is common to measure both axial and diametral strain during compression to determine elastic modulus and Poisson's ratio. Test procedures are given in ASTM Standard (D3148) and discussed further by Wyllie (1999). It is important to note that the ASTM procedure defines several methods of determination of modulus, including tangent modulus at a specified stress level, average modulus over the

TABLE 7
COMMON LABORATORY TESTS FOR INTACT ROCK

Test Category	Name of Test and ASTM Designation	Comments
Uniaxial compression	Unconfined compressive strength of intact rock core specimen (D2938)	Primary test for strength and deformability of intact rock; input parameter for rock mass classification systems
Split tensile	Splitting tensile strength of intact rock core specimens (D3967)	Splitting tensile strength of a rock disk under a compression line load
Point load strength	Determination of the point load strength index of rock (D5731)	Index test for rock strength classification; can be performed in field on core pieces unsuitable for lab testing
Direct shear	Laboratory direct shear strength tests for rock specimens under constant normal stress (D5607)	Applies to intact rock strength or to shear strength along planes of discontinuities, including rock–concrete interface
Strength-deformation	Elastic moduli of intact rock core specimens in uniaxial compression (D3148)	Young's modulus from axial stress–strain curve; Poisson's ratio can also be determined
Durability	Slake durability of shales and similar weak rocks (D4644)	Index test to quantify the durability of weak rocks under wetting and drying cycles with abrasion

linear portion of the stress–strain curve, and secant modulus at a fixed percentage of maximum strength. For rocks that exhibit nonlinear stress–strain behavior, these methods may provide significantly different values of modulus and it is important to note which method was used when reporting values of modulus.

The point load test is conducted by compressing a core sample or irregular piece of rock between hardened steel cones (Figure 13), causing failure by the development of tensile cracks parallel to the axis of loading. The uncorrected point load strength index is given by

$$I_s = P/D^2 \tag{4}$$

where P = load at rupture, and D is the distance between the point loads. The point load index is reported as the point load strength of a 50 mm core. For other specimen sizes a correction factor is applied to determine the equivalent strength of a 50 mm specimen. The point load index is correlated to uniaxial compressive strength by

$$q_u = C\, I_{s(50)} \tag{5}$$

where q_u is the unconfined compressive strength, $I_{s(50)}$ is the point load strength corrected to a diameter of 50 mm, and

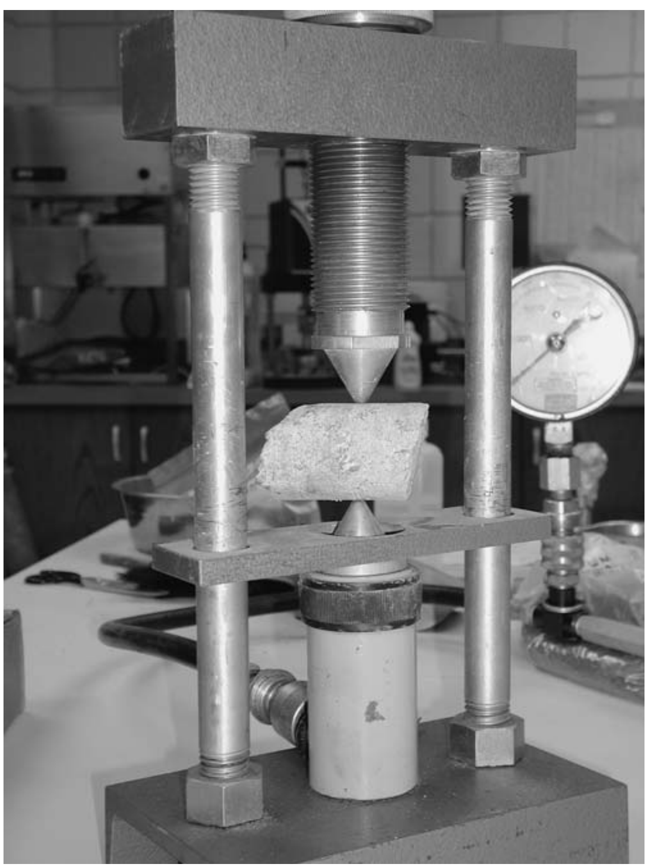


FIGURE 13 Point load test setup.

C is a correlation factor that should be established on a site-specific basis by conducting a limited number of uniaxial compression tests on prepared core samples. If a site-specific value of C is not available, the ASTM Standard recommends approximate values based on core diameter. For a 54 mm core (NX core size), the recommended value of C is 24. The principal advantages of the point load test are that it can be carried out quickly and inexpensively in the field at the site of drilling and that tests can be conducted on irregular specimens without the preparation required for uniaxial compression tests.

Split tensile strength (q_t) of rock (ASTM D4644) is determined by compressing a cylindrical disk under a compressive line load. Split tensile strength has been correlated with unit side resistance; for example, by McVay et al. (1992) for drilled shafts in Florida limestone.

Direct shear testing is applicable to determination of the Mohr–Coulomb shear strength parameters cohesion, c, and friction angle, ϕ, of discontinuity surfaces in rock (ASTM D5607). Shear strength of discontinuities may govern capacity in certain conditions; for example, base capacity of socketed foundations when one or two intersecting joint sets are oriented at an intermediate angle to horizontal. The other notable application of this test is in simulating the shear strength at the rock–concrete interface for evaluation of side resistance of socketed shafts under axial loading. However, for this application, the constant normal stiffness (CNS) direct shear test described by Johnston et al. (1987) is more applicable. Instead of a constant normal load, normal force is applied through a spring that increases or decreases the applied force in proportion to the magnitude of normal displacement (dilation). Dilatancy of the interface is a major factor controlling strength and stiffness of socketed shafts under axial load.

The slake durability test (ASTM D4644) provides an index for identifying rocks that will weather and degrade rapidly. The test is appropriate for argillaceous sedimentary rocks (mudstone, shale, clay–shales) or any weak rock. Representative rock fragments are placed in a wire mesh drum and dried in an oven to constant weight. The drum is partially submerged in water and rotated at 20 revolutions per minute for a period of 10 min. The drum and its contents are then dried a second time and the loss of weight is recorded. The test cycle is repeated a second time and the slake durability index, I_D, is calculated as the ratio (reported as a percentage) of final to initial dry weights of the sample. Rocks with $I_D < 60$ are considered prone to rapid degradation and may indicate a susceptibility to degradation of the borehole wall when water is introduced during drilling, potentially leading to formation of a "smear zone." Hassan and O'Neill (1997) define the smear zone as a layer of soil-like material along the socket wall and demonstrate that smearing can have a significantly negative effect on side load transfer of shafts in argillaceous rock.

In Situ Tests for Rock

In situ testing can be used to evaluate rock mass deformation modulus and, in some instances, RMS. In situ testing methods with potential applications to rock-socket design are presented in Table 8. In situ testing of rock is not performed routinely for rock-socket design by most of the agencies surveyed for this study. The survey responses indicate that five state DOTs currently use the pressuremeter test (PMT) to obtain design parameters. Of these, all five use the test to obtain rock mass modulus. One state reported the use of PMT to evaluate RMS in weak rocks. Four states use the PMT for correlating test results with the parameters that define *p-y* curves for analysis of shafts under lateral loading (chapter four). The term dilatometer is also used to describe a pressuremeter intended for use in rock but should not be confused with the flat plate dilatometer used for in situ testing of soil. One state (Massachusetts) reported using the borehole jack to measure rock mass modulus. No states reported using the plate load test for rock-socket design. Information on conduct and interpretation of the tests identified in Table 8 and other in situ tests for rock are given in the relevant ASTM standards, *Rock Testing Handbook* (1993) and Mayne et al. (2001).

Heuze (1980) investigated the effect of test scale on the modulus of rock masses. Several types of field tests, including borehole jack and plate load tests at different scales, were included and results were compared with those of laboratory compression tests. It was observed that in situ rock mass modulus values generally range from 20% to 60% of intact rock modulus from laboratory uniaxial compression tests. The borehole jack was recommended as a field test that, with proper analysis (Heuze 1984), yields values of rock mass modulus that are consistent with results from large plate bearing tests. The borehole jack designed for NX sized borings (75 mm or 3 in. diameter) affects a "test volume" of approximately 0.14 m^3 (5 ft^3). Borehole jack devices are available commercially with limit pressures of up to 69 MPa, allowing the test to reach stress levels beyond the elastic limit and, for some weak rock masses, to ultimate strength.

Studies on the use of PMTs for determination of rock mass modulus include those of Rocha et al. (1970), Bukovansky (1970), Georgiadis and Michalopoulos (1986), and Littlechild et al. (2000). Results have been mixed, with some researchers indicating a high degree of agreement between PMT modulus and other in situ tests (e.g., Rocha et al. 1970) and others reporting PMT modulus values significantly lower than modulus measured by plate-load and borehole jack tests (e.g., Bukovansky 1970). Littlechild et al. (2000) concluded that PMTs, using the Cambridge High Pressure Dilatometer, were not useful for determination of rock mass modulus for design of deep foundations in several rock types in Hong Kong. In strong and massive rocks such as metasiltstone and tuff, the device did not have sufficient capacity to measure modulus, which typically was around 10 GPa. In highly fractured granodiorite, membrane failures were problematic. Commercially available pressuremeter devices for rock are currently limited to maximum pressures of around 30 MPa. Additional discussion of rock mass modulus is presented later in this chapter.

TABLE 8
IN SITU TESTS WITH APPLICATIONS TO ROCK-SOCKET DESIGN

Method	Procedure	Rock Properties	Limitations/Remarks
Pressuremeter (includes devices referred to as rock dilatometer)	Pressuremeter is lowered to the test elevation in a prebored hole; flexible membrane of probe is expanded exerting a uniform pressure on the sidewalls of the borehole	Rock mass modulus; rock mass strength in weak rocks ASTM D4719	Test affects a small area of rock mass; depending on joint spacing, may or may not represent mass behavior; limited to soft or weak rocks
Borehole jack	Jacks exert a unidirectional pressure to the walls of a borehole by means of two opposed curved steel platens	Rock mass modulus; rock mass strength in weak rocks ASTM D4971	Measured modulus value must be corrected to account for stiffness of steel platens; test method can be used to provide an estimate of anisotropy
Plate load test	Load is applied to a steel plate or concrete foundation using a system of hydraulic jacks and a reaction frame anchored to the foundation rock	Rock mass modulus; rock mass strength in weak rocks	Loaded area is limited, so may not be effectively testing rock mass if joints are widely spaced; modulus values corrected for plate geometry, effect of rock breakage, rock anisotropy, and steel plate modulus; not common for deep foundations
Texas cone penetration test	Steel cone is driven by a drop hammer; number of blows per 300 mm of penetration is TCPT *N*-value; depth of penetration per 100 blows is penetration resistance (PR)	Correlated to compressive strength of weak rocks encountered in Texas and Oklahoma	Limitations similar to those of Standard Penetration Test; currently used by Texas and Oklahoma DOTs for direct correlation to side and base resistance of shafts in weak rock

Notes: Adapted from *Geotechnical Engineering Circular No. 5* (Sabatini et al. 2002).
TCPT = Texas Cone Penetration Test.

An example of an in situ test that is used in a specific region of the country is the Texas Cone Penetration Test (TCPT). A 76-mm-diameter solid steel cone is driven by a 77 kg (170 lb) drop hammer. The number of blows required to drive 300 mm (12 in.) is recorded and the results are given in one of two ways: (1) number of blows per 300 mm of penetration or TCPT *N*-value, or (2) the depth of penetration per 100 blows, referred to as the penetration resistance or *PR*. The Texas and Oklahoma DOTs use empirical correlations between the TCPT parameters and drilled shaft side and base resistances in soil and soft rock. The test procedure and correlations are available in the Texas DOT *Geotechnical Manual*, which can be accessed online. Some researchers have developed empirical correlations between TCPT measurements and properties of soft rock. For example, Cavusoglu et al. (2004) show correlations between compressive strength of upper Cretaceous formation clay shales (UU triaxial tests) and limestone (unconfined compression) and *PR* measurements conducted for Texas DOT projects. The correlations are highly formation-dependent and exhibit a high degree of scatter, but provide first order estimates of rock strength based on TCPT resistance in formations where sample recovery is otherwise difficult.

In addition to the tests identified as being applicable to rock, it is common practice to use in situ tests for soil to define the contact boundary between soil and rock. Of the agencies surveyed, 21 reported using the Standard Penetration Test (SPT) and 3 reported using the Cone Penetration Test (CPT) to define the top-of-rock elevation. "Refusal" of the SPT or CPT penetration is the method most often used to identify rock. Limitations of this approach include the possibility of mistaking cobbles or boulders for the top-of-rock and the lack of consistency in SPT blowcounts in weak or weathered rock.

Six states reported using the SPT in soft or weak rock to obtain rock properties (unconfined compressive strength) or for correlating SPT *N*-values directly to design parameters, principally unit side resistance. For example, the Colorado SPT-Based Method is used by the Colorado DOT to establish design values of both unit side resistance and base resistance for shafts socketed into claystones when the material cannot be sampled in a way that provides intact core specimens adequate for laboratory uniaxial compression tests (Abu-Hejleh et al. 2003). O'Neill and Reese (1999) correlate unit side resistance with *N*-values for shafts in cohesionless IGMs, defined as materials with $N > 50$. Direct correlations between design parameters and *N* values are considered further in chapter three.

Rock Mass Classification

Several empirical classification systems have been proposed for the purpose of rating rock mass behavior. The most widely used systems are the Geomechanics Classification described by Bieniawski (1976, 1989) and the Rock Quality Tunneling Index described by Barton et al. (1974). Both systems were developed primarily for application to tunneling in rock, but have been extended to other rock engineering problems. The application of classification systems to rock-socket design has been limited to correlations between classification parameters and RMS and deformation properties. To facilitate such correlations, Hoek et al. (1995) introduced the GSI. Relationships were developed between GSI and the rock mass classifications of Bieniawski and Barton et al. The principal characteristics of the two classification systems are summarized, followed by a description of their relationship to GSI. For more detailed discussion, including limitations and recommended applications, consult the original references and Hoek et al. (1995, 2002).

The Geomechanics Classification is based on determination of the RMR, a numerical index determined by summing the individual numerical ratings for the following five categories of rock mass parameters:

- Strength of intact rock,
- Drill core quality (in terms of RQD),
- Spacing of discontinuities,
- Condition of discontinuities, and
- Groundwater conditions.

An adjustment is made to the RMR for the degree to which joint orientation may be unfavorable for the problem under consideration. The classification system is presented in Table 9. Based on the RMR value, a rock mass is identified by one of five rock mass classes, ranging from very poor rock to very good rock. The draft 2006 Interim AASHTO *LRFD Bridge Design Specifications* recommends determination of RMR for classification of rock mass in foundation investigations. Seventeen states reported using RMR either always or sometimes for rock mass classification associated with drilled shaft design.

Barton and co-workers at the Norwegian Geotechnical Institute proposed a Tunneling Quality Index (Q) for describing rock mass characteristics and tunnel support requirements (Barton et al. 1974). The system is commonly referred to as the NGI-Q system or simply the Q-system. The numerical value of the index Q varies on a log scale from 0.001 to 1,000 and is defined as:

$$Q = \frac{\mathrm{RQD}}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{\mathrm{SRF}} \tag{6}$$

where
RQD = rock quality designation,
J_n = joint set number,
J_r = joint roughness number,
J_a = joint alteration number,
J_w = joint water reduction factor, and
SRF = stress reduction factor.

TABLE 9
GEOMECHANICS CLASSIFICATION SYSTEM FOR DETERMINATION OF ROCK MASS RATING (RMR)

A. Classification Parameters and Their Ratings (after Bieniawski 1989)

	Parameter		Ranges of Values				
1	Strength of intact rock material	Point load strength index, MPa	>10	4–10	2–4	1–2	For this low range, uniaxial comp. test is preferred
		Uniaxial comp. strength, MPa	>250	100–250	50–100	25–50	5–25 / 1–5 / <1
	Rating		15	12	7	4	2 / 1 / 0
2	Drill core quality, RQD (%)		90–100	75–90	50–75	25–50	<25
	Rating		20	17	13	8	3
3	Spacing of discontinuities		>2 m	0.6–2 m	200–600 mm	60–200 mm	<60 mm
	Rating		20	15	10	8	5
4	Condition of discontinuities		Very rough surfaces, not continuous, no separation, unweathered wall rock	Slightly rough surfaces, separation <1 mm, slightly weathered walls	Slightly rough surfaces, separation <1 mm, highly weathered walls	Slickensided surfaces or gouge <5 mm thick *or* joints open 1 to 5 mm continuous	Soft gouge >5 mm thick or separation >5 mm continuous
	Rating		30	25	20	10	0
5	Ground-water	Inflow per 10 m tunnel length	None	<10	10–25	25–125	>125
		Ratio: Joint water pressure/ major principal stress	0	<0.1	0.1–0.2	0.2–0.5	>0.5
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing
	Rating		15	10	7	4	0

B. Rating Adjustment for Joint Orientations

Strike and dip orientations		Very favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Foundations	0	–2	–7	–15	–25

C. Rock Mass Classes Determined from Total Ratings

RMR	100 to 81	80 to 61	60 to 41	40 to 21	<20
Class Number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

Three states reported using the Q-system in connection with rock-socket design. A modified Tunneling Quality Index (Q') is utilized to determine the GSI, as described subsequently.

The Geomechanics Classification can be used to estimate the value of GSI for cases where RMR is greater than 23, as follows:

$$\mathrm{GSI} = \mathrm{RMR}_{89} - 5 \tag{7}$$

in which RMR_{89} is the RMR according to Bieniawski (1989) as presented in Table 9. For RMR_{89} values less than 23, the modified (Q') is used to estimate the value of GSI, where:

$$Q' = \frac{\mathrm{RQD}}{J_n} \times \frac{J_r}{J_a} \tag{8}$$

$$\mathrm{GSI} = 9\mathrm{Log}_e Q' + 44 \tag{9}$$

Table 10 gives the values of the parameters used to evaluate Q' by Eq. 8.

Engineering Properties of Rock Mass

Shear Strength

Geotechnical evaluation of foundation ultimate capacity under axial and lateral loading is calculated on the basis of shear strength along assumed failure surfaces in the rock or at the concrete–rock interface. Depending on the failure mode, the strength may need to be defined at one of three levels: (1) intact rock, (2) along a discontinuity, and (3) representative of a highly fractured rock mass. Figure 14 illustrates these cases for a socketed foundation in rock. For example, bearing

TABLE 10
JOINT PARAMETERS USED TO DETERMINE Q'

1. No. of Sets of Discontinuities = J_n	
Massive	0.5
One set	2
Two sets	4
Three sets	9
Four or more sets	15
Crushed rock	20

2. Roughness of Discontinuities = J_r	
Noncontinuous joints	4
Rough, wavy	3
Smooth, wavy	2
Rough, planar	1.5
Smooth, planar	1
Slick and planar	0.5
Filled discontinuities	1

3. Discontinuity Condition & Infilling = J_a	
3.1 Unfilled cases	
Healed	0.75
Stained, no alteration	1
Silty or sandy coating	3
Clay coating	4
3.2 Filled discontinuities	
Sand or crushed rock infill	4
Stiff clay infilling <5 mm	6
Soft clay infill <5 mm thick	8
Swelling clay <5 mm	12
Stiff clay infill >5 mm thick	10
Soft clay infill >5 mm thick	15
Swelling clay >5 mm	20

*Note: Add +1 if mean joint spacing > 3 m. Modified from Barton et al. (1974).

capacity at the base of a socketed foundation in massive rock would be evaluated in terms of the strength of the intact rock. If the rock has regular discontinuities oriented as shown in level 2, base capacity may be controlled by the strength along the joint surfaces. If the rock is highly fractured (level 3), bearing capacity would have to account for the overall strength of the fractured mass.

For each of the three cases, shear strength may be expressed within the framework of the Mohr–Coulomb failure criterion, where shear strength (τ) is given by

$$\tau = c' + \sigma' \tan \phi' \qquad (10)$$

in which c' = effective stress cohesion intercept, ϕ' = effective stress angle of friction, and σ' = effective normal stress on the failure plane. Evaluation of shear strength for each of the three cases is summarized as follows.

For intact rock the parameters c' and ϕ' can be determined from laboratory triaxial shear tests on specimens prepared from core samples. Triaxial testing procedures are given by ASTM D2664 and AASHTO T226. The survey of state DOTs indicates that triaxial testing is not used routinely. The most common test for intact rock is the uniaxial (unconfined) compression test, which can be considered a special case of triaxial testing with zero confining stress. The strength parameter obtained is the uniaxial compressive strength, q_u, which is related to the Mohr–Coulomb strength parameters by

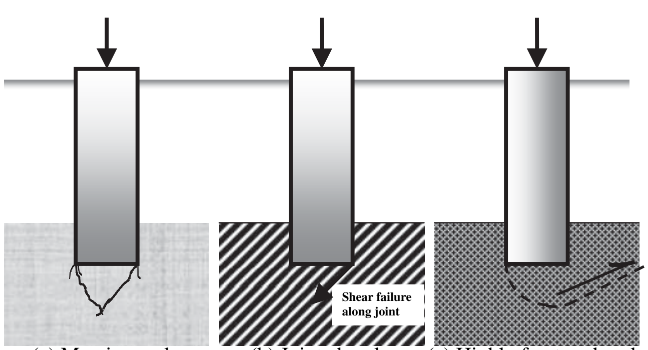
$$q_u = 2c \tan (45^\circ + \tfrac{1}{2} \phi) \qquad (11)$$

However, the strength of intact rock is normally given simply in terms of q_u. Stability analyses of rock sockets governed by massive rock are normally evaluated directly in terms of q_u. When rock core is not sufficient for uniaxial compression testing, or sometimes for convenience, q_u is correlated to results of point load tests. Uniaxial compressive strength is also one of the parameters used for evaluating the strength of highly fractured rock masses, as discussed later.

Shear strength of discontinuities can be determined using laboratory direct shear tests. The apparatus is set up so that the discontinuity surface lies in the plane of shearing between the two halves of the split box. Both peak and residual values of the strength parameters (c' and ϕ') are determined. Discussion of direct shear testing of discontinuities, including its limitations, is given by Wyllie and Norrish (1996).

For a planar, clean fracture (no infilling), the cohesion is zero and the shear strength is defined only by the friction angle. The roughness of the surface has a significant effect on the value of friction angle. If the discontinuity contains infilling, the strength parameters will be controlled by the thickness and properties of the infilling material. Compilations of typical representative ranges of strength parameter values for discontinuities are summarized in Mayne et al. (2001). The survey results indicate that direct shear testing of joints is *not* conducted routinely by DOT agencies for rock-socket design.

For intact rock masses and for fractured or jointed rock masses, Hoek and Brown (1980) proposed an empirical criterion for characterizing RMS. Since its appearance, this criterion has been applied widely in practice and considerable experience



(a) Massive rock (b) Jointed rock (c) Highly fractured rock

FIGURE 14 Base failure modes illustrating different operational shear strength conditions.

has been gained for a range of rock engineering problems. Based on these experiences, the criterion has undergone several stages of modification, most significantly by Hoek and Brown (1988), Hoek et al. (1995, 2002), and Marinos and Hoek et al. (2000). The nonlinear RMS is given by:

$$\sigma_1' = \sigma_3' + q_u \left(m_b \frac{\sigma_3'}{q_u} + s \right)^a \tag{12}$$

where

σ'_1 and σ'_3 = major and minor principal effective stresses, respectively;
q_u = uniaxial compressive strength of intact rock; and
m_b, s, and a are empirically determined strength parameters for the rock mass.

The value of the constant m for intact rock is denoted by m_i and can be estimated from Table 11. Hoek and Brown (1988) suggested that the constants m_b, s, and a could be related empirically to the RMR described previously. Hoek et al. (1995) noted that this process worked well for rock masses with RMR greater than about 25, but not well for very poor rock masses. To overcome this limitation, the GSI was introduced. Suggested relationships between GSI and the parameters m_b/m_i, s, and a, according to Hoek et al. (2002) are as follows:

$$\frac{m_b}{m_i} = \exp\left(\frac{\text{GSI} - 100}{28 - 14D} \right) \tag{13}$$

$$s = \exp\left(\frac{\text{GSI} - 100}{9 - 3D} \right) \tag{14}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-\text{GSI}}{15}} - e^{\frac{-20}{3}} \right) \tag{15}$$

in which D is a factor that depends on the degree of disturbance to the rock mass caused by blast damage and stress relaxation.

TABLE 11
VALUES OF THE CONSTANT m_i BY ROCK GROUP (Hoek et al. 1995)

Rock Type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
Sedimentary	Clastic		Conglomerate (22)	Sandstone 19	Siltstone 9	Claystone 4
				<------------ Graywacke (18)	-------------->	
	Non-clastic	Organic		<--------------- Chalk 7	----------------->	
				<----------------- Coal (8–21)	----------------->	
		Carbonate	Breccia (20)	Sparitic limestone (10)	Micritic limestone 8	
		Chemical		Gypstone 16	Anhydrite 13	
Metamorphic	Non-foliated		Marble 9	Hornfels (19)	Quartzite 24	
	Slightly foliated		Migmatite (30)	Amphibolite 31	Mylonites (6)	
	Foliated*		Gneiss 33	Schists (10)	Phyllites (10)	Slate 9
Igneous	Light		Granite 33		Rhyolite (16)	Obsidian (19)
			Granodiorite (30)		Dacite (17)	
			Diorite (28)		Andesite 19	
	Dark		Gabbro 27	Dolerite (19)	Basalt (17)	
			Norite 22			
	Extrusive pyroclastic type		Agglomerate (20)	Breccia (18)	Tuff (15)	

*These values are for intact rock specimens tested normal to foliation. The value of m_i will be significantly different if failure occurs along a foliation plane.
Note: Values in parentheses are estimates.

The damage factor D ranges from zero for undisturbed in situ rock masses to 1.0 for very disturbed rock masses. Hoek et al. (2002) provide guidance on values of D for application to tunnel and rock slope problems, but no work has been published relating D to drilled shaft construction.

Some problems involving fractured rock masses (e.g., bearing capacity) are more readily analyzed in terms of the Mohr–Coulomb strength parameters than in terms of the Hoek–Brown criterion. Hoek and Brown (1997) noted that there is no direct correlation between the two sets of strength parameters. However, they describe a procedure that involves simulating a set of triaxial strength tests using the Hoek–Brown criterion (Eq. 12) then fitting the Mohr–Coulomb failure envelope to the resulting Mohr's circles by regression analysis. Values of the strength parameters c' and ϕ' defining the intercept and tangent slope of the envelope (which is nonlinear) can thus be determined. Hoek et al. (2002) presented the following equations for the angle of friction and cohesive strength of fractured rock masses:

$$\phi' = \sin^{-1}\left[\frac{6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}{2(1+a)(2+a)+6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}\right] \tag{16}$$

$$c' = \frac{q_u\left[(1+2a)s+(1-a)m_b\sigma'_{3n}\right]\left(s+m_b\sigma'_{3n}\right)^{a-1}}{(1+a)(2+a)\sqrt{\dfrac{1+6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}{(1+a)(2+a)}}} \tag{17}$$

Applications of the Hoek–Brown criterion to rock-socket design are discussed further in chapter three (bearing capacity) and chapter four (lateral capacity). The draft 2006 Interim AASHTO *LRFD Bridge Design Specifications* recommend the Hoek–Brown strength criterion for RMS characterization, but the earlier version (Hoek and Brown 1988) is presented rather than the updated version based on GSI.

Deformation Properties

Rock mass deformation properties are used in analytical methods for predicting the load-deformation behavior of rock-socketed foundations under axial and lateral loads. The parameters required by most methods include the modulus of deformation of the rock mass, E_M, and Poisson's ratio, v. Methods for establishing design values of E_M include:

- Estimates based on previous experience in similar rocks or back-calculated from load tests,
- Correlations with seismic wave velocity propagation (e.g., Eqs. 1–3),
- In situ testing, and
- Empirical correlations that relate E_M to strength or modulus values of intact rock (q_u or E_R) and/or rock mass characteristics.

Compilations of typical values of rock mass modulus and Poisson's ratio are given in several sources, including Kulhawy (1978), Wyllie (1999), and the AASHTO *LRFD Bridge Design Specifications* (2004). These values should be considered as general guidelines to expected ranges of values for different rock types and serve to illustrate the magnitude of variation that is possible. Rock mass modulus can vary from less than 1 MPa to greater than 100 GPa and depends on intact rock modulus, degree of weathering, and characteristics of discontinuities. Compiled values provide guidance for very preliminary evaluations, but should not be relied on for final design. Values of Poisson's ratio exhibit a narrow range of values, typically between 0.15 and 0.3.

Various authors have proposed empirical correlations between rock mass modulus and other rock mass properties. Table 12 presents, in chronological order, some of the most widely cited expressions found in the literature. The earliest published correlations (expressions 1 and 2 of Table 12) relate E_M to modulus of intact rock, E_R, and RQD. In subsequent correlations (expression 3), RQD is replaced by RMR, providing a more comprehensive empirical approach because six rock mass parameters (including RQD) are incorporated to evaluate the RMR. This was followed by correlations relating E_M directly to rock mass indexes, including *RMR* and *Q* (expressions 4, 5, and 6). Hoek et al. (1995) show the graph given in Figure 15 with curves given by expressions 4, 5, and 6 of Table 12, along with case history observations. The figure suggests that expression 4 of Table 12 provides a reasonable fit to the available data and offers the advantage of covering a wider range of RMR values than the other equations. The draft 2006 Interim AASHTO *LRFD Bridge Design Specifications* recommend use of either expression 4 of Table 12 or a method recommended by O'Neill et al. (1996) based on applying a modulus reduction ratio (E_M/E_R) given as a function of RQD in Table 13.

Beginning with Hoek and Brown (1997), proposed correlation equations have been based on relating E_M to GSI and properties of intact rock, either uniaxial compressive strength (q_u) or intact modulus (E_R). In expression 7, E_M is reduced progressively as the value of q_u falls below 100 MPa. This reduction is based on the reasoning that deformation of better quality rock masses is controlled by discontinuities, whereas for poorer quality rock masses deformation of the intact rock pieces contributes to the overall deformation process (Hoek and Brown 1997). The version given in Table 12 is updated by Hoek et al. (2002) to incorporate the damage factor, D.

The final correlation (expression 8) in Table 12 was proposed based on analyses by Yang (2006). Figure 16 shows a comparison of the regression equation (expression 8) to data from field observations of Bieniawski (1978) and Serafim and Pereira (1983), as well as modulus values measured by PMTs reported by Yang (2006). Expression 8 was applied to derivation of *p-y* curves for analysis of laterally loaded rock sockets, described further in chapter four. Additional discussion of empirical equations for rock mass modulus and their

TABLE 12
EMPIRICAL METHODS FOR ESTIMATING ROCK MASS MODULUS

Expression	Notes/Remarks	Reference
1. $E_M = E_R[0.0231(\text{RQD}) - 1.32]$	Reduction factor on intact rock modulus; $E_M/E_R \geq 0.15$	Coon and Merritt (1969); *LRFD Bridge Design* . . . (2004)
2. For RQD < 70: $E_M = E_R$ (RQD/350) For RQD > 70: $E_M = E_R$ [0.2 + (RQD – 70)/37.5]	Reduction factor on intact rock modulus	Bieniawski (1978)
3. $E_M = E_R\left[0.1+\dfrac{\text{RMR}}{1150-11.4\text{RMR}}\right]$	Reduction factor on intact rock modulus; $E_M/E_R \leq 1.0$	Kulhawy (1978)
4. $E_M\,(\text{GPa}) = 10^{\frac{\text{RMR}-10}{40}}$	$0 < \text{RMR} < 90$	Serafim and Pereira (1983)
5. E_M (GPa) = 2 RMR – 100	$45 < \text{RMR} < 90$	Bieniawski (1984)
6. E_M (GPa) = 25 $\log_{10} Q$	$1 < Q < 400$	Hoek et al. (1995)
7. $E_M\,(\text{GPa}) = \left(1-\dfrac{D}{2}\right)\sqrt{\dfrac{q_u}{100}}\,10^{\frac{\text{GSI}-10}{40}}$ for $q_u \leq 100$ MPa $E_M\,(\text{GPa}) = \left(1-\dfrac{D}{2}\right)10^{\frac{\text{GSI}-10}{40}}$ for $q_u > 100$ MPa	Adjustment to Serafim and Pereira to account for rocks with $q_u < 100$ MPa; *note* q_u in MPa	Hoek and Brown (1997); Hoek et al. (2002)
8. $E_M = \dfrac{E_R}{100}e^{\text{GSI}/21.7}$	Reduction factor on intact modulus, based on GSI	Liang and Yang (2006)

Notes: E_R = intact rock modulus, E_M = equivalent rock mass modulus, RQD = rock quality designation, RMR = rock mass rating, Q = NGI rating of rock mass, GSI = geological strength index, q_u = uniaxial compressive strength.

application to foundation engineering is given by Littlechild et al. (2000), Gokceoglu et al. (2003), and Yang (2006).

Rock mass modulus is a key parameter for rock-socket load-deformation analysis, which is a key step in the design process depicted in Figure 3. Several methods are identified in this chapter for establishing values of E_M. These include geophysical methods based on p-wave and s-wave velocities (Eqs. 1 and 2) or shear wave frequency (Eq. 3), in situ testing methods (Table 8), and the correlation equations given in Table 12. The survey shows that correlation equations are the most widely used method for estimating modulus for rock-socket design, followed by in situ testing. The most common in situ test (used by five states) is pressuremeter (rock dilatometer), with a single state (Massachusetts) reporting use of the borehole jack test. At least three other states using PMT for rock did not respond to the survey. The principal limitation of in situ testing is whether the volume of rock being tested is representative of the in situ rock mass. Factors such as degree of rock disturbance, anisotropy, and spacing of discontinuities relative to the dimensions of the apparatus will determine the degree to which test results represent the response of rock mass to foundation loading. As noted earlier in this chapter, rock mass modulus measured by pressuremeter shows varying levels of agreement with other in situ testing methods. The full range of application and limitations of PMTs for rock mass modulus and its application to rock-socket design have yet to be determined. Correlation equations for rock mass modulus have evolved over the years as illustrated by the relationships summarized in Table 12. Correlations are attractive because they are based on more easily measured properties of intact rock and rock mass indexes, but caution must be exercised because most of the correlations were developed specifically for applications to tunneling. Calibration studies aimed at the application of correlation equations for rock mass modulus to load-deformation analysis of rock-socketed foundations are largely lacking at the present time. Studies by Littlechild et al. (2000) and Liang and Yang (2006) are exceptions and illustrate the type of additional work that is needed.

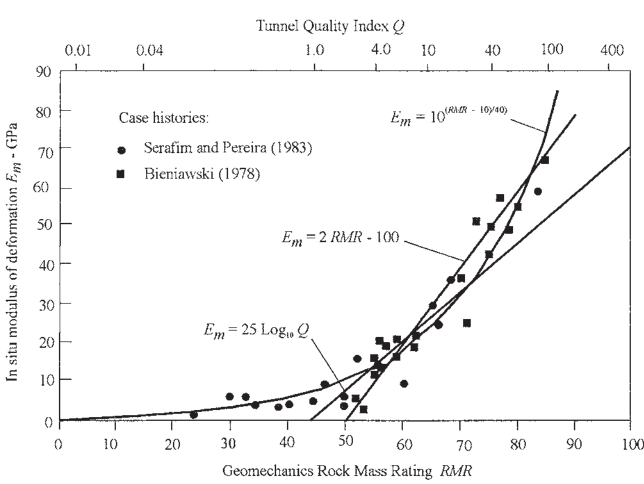


FIGURE 15 Rock mass modulus versus rock mass rating (Hoek et al. 1995).

TABLE 13
ESTIMATION OF MODULUS RATIO (E_M/E_R) BASED ON RQD (O'Neill et al. 1996)

	E_M/E_R	
RQD (percent)	Closed Joints	Open Joints
100	1.00	0.60
70	0.70	0.10
50	0.15	0.10
20	0.05	0.05

RQD = rock quality designation.

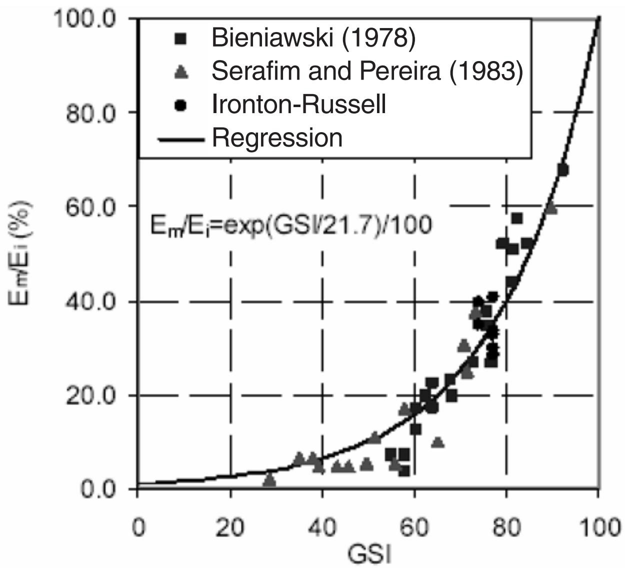


FIGURE 16 Ratio of rock mass modulus to modulus of intact rock versus Geological Strength Index (Yang 2006).

A case history described by LaFronz et al. (2003) illustrates the use of multiple methods for establishing design values of rock mass modulus. Site characterization for the Colorado River Bridge (Hoover Dam Bypass Project) included borehole jack, downhole seismic (compression wave velocity), and laboratory uniaxial compression tests. The major rock unit for the abutment foundations on the Arizona side of the bridge is Hoover Dam tuff (welded volcanic ash). Results of field and laboratory tests used to establish rock mass modulus in the tuff are summarized in Table 14. Values given for the correlation with GSI reflect two values of GSI for the tuff, one corresponding to fracture conditions of width = 1 to 5 mm with soft filling (GSI = 45) and the other corresponding to fracture width of 0.1 to 1 mm and no filling (GSI = 52). Modulus values based on downhole *p*-wave velocities were calculated using equations given by Viskne (1976), described by LaFronz et al. (2003) as valid at the rock mass scale.

Results were applied as follows. Borehole jack measured values at stress ranges representative of expected footing bearing pressures were taken as reasonable values for developing foundation load-deflection curves. Deformation modulus predicted by the correlation to GSI (Table 12, Hoek and Brown 1997) provided a cross-check on the borehole jack measured values. The mean value of modulus from the borehole jack tests is in the range of the GSI-predicted values. A low-strain modulus derived from downhole seismic measurements was used as a reasonable upper-bound check on the rock mass modulus. The modulus of intact rock from laboratory uniaxial compression tests on core samples is consistent with the observation of Heuze (1980) that field rock mass modulus values range from 20% to 60% of intact rock modulus and serve as an additional upper-bound check.

TABLE 14
MODULUS VALUES, HOOVER DAM TUFF
(LaFronz et al. 2005)

Method	Mean Modulus (GPa)
Borehole jack	2.83
Correlation to GSI	2.34, 3.52
Downhole seismic	3.31
Uniaxial compression	13.79

INTERMEDIATE GEOMATERIALS

A persistent challenge to the geotechnical engineer, and one that pertains directly to design and construction of drilled shafts, is defining the boundary between soil and rock. Different approaches to site characterization and evaluation of geomaterial properties and different design methods are used when the geomaterial involved is clearly defined as soil or as rock. However, many geomaterials encountered in practice exhibit properties that make it difficult to define them clearly as being soil or rock within the context of standardized classification systems. Geologic processes provide us with a continuum of geomaterial properties and characteristics, some of which defy simplified categorization.

The term intermediate geomaterial (IGM) has been applied recently to earth materials with properties that are at the boundary between soil and rock (O'Neill et al. 1996). The criteria are based on (1) whether the material is cohesionless or cohesive and (2) some index of material strength. Cohesionless IGMs are defined by O'Neill et al. (1996) as very dense granular geomaterials, such as residual, completely decomposed rock and glacial till, with SPT N_{60}-values between 50 and 100. Cohesive IGMs are defined as materials that exhibit unconfined compressive strengths in the range of 0.5 MPa $\leq q_u \leq$ 5 MPa. Specific materials identified by O'Neill et al. (1996) as being cohesive IGMs include (1) argillaceous geomaterials, such as heavily overconsolidated clays, clay shales, saprolites, and mudstones that are prone to smearing when drilled; and (2) calcareous rocks such as limestone and limerock and argillaceous geomaterials that are not prone to smearing when drilled. The term IGM as used by O'Neill et al. (1996) and subsequently adopted in O'Neill and Reese (1999) and in the draft 2006 Interim AASHTO *LRFD Bridge Design Specifications* has been limited specifically to design of drilled shafts and has not been adopted in the general geotechnical literature. For example, the term IGM is not used in the FHWA *Manual on Subsurface Investigations* (Mayne et al. 2001) or in "Evaluation of Soil and Rock Properties," *Geotechnical Engineering Circular No. 5* (Sabatini et al. 2002). Responses to Question 8 of the survey show that most responding states (23) define IGMs for drilled shaft design according to the criteria of O'Neill et al. (1996). However, six states responded that geomaterials are classified as either soil or rock and IGM is not used.

According to O'Neill and Reese (1999) cohesionless IGMs may be treated, for practical purposes, in the same manner as coarse-grained (cohesionless) soils. They are assumed to respond to loading by rapid dissipation of excess pore water pressure (fully drained response) and are analyzed within the context of effective stress. For strength analysis, cohesionless IGMs are characterized in terms of the effective stress angle of friction ϕ'. It should be noted that some empirical correlations that apply to cohesionless soils, such as friction angle estimated from SPT *N*-values, may not be applicable to cohesionless IGMs. Specific approaches for estimating design parameters of shafts in cohesionless IGM are covered in chapter three.

The definition of cohesive IGMs given earlier is based on a single index, the unconfined compressive strength. Although this categorization may be useful to identify materials falling into a defined range of intact strength, it does not necessarily provide the distinction between soil and rock most relevant to behavior of drilled shafts. To illustrate, consider Figure 17 from Kulhawy and Phoon (1993). This figure shows the relationship between unit side resistance determined from field load tests on drilled shafts and one-half of the unconfined compressive strength. Both parameters are normalized by atmospheric pressure p_a. Two categories of load tests were defined; those conducted on shafts in fine-grained soils (clay) and those in rock. Kulhawy and Phoon relied on the judgment of the original authors and the database compilers to establish whether the material was soil or rock. For convenience, the range of normalized strength that defines cohesive IGM is superimposed on Figure 17. It can be seen that the soil and rock data constitute apparently different populations, including over the range of strength that defines cohesive IGM. For purposes of drilled shaft side resistance, therefore, the classification of IGM does not provide a smooth transition from soil to rock. It may be more meaningful to define the material as being one or the other on the basis of additional geologic information.

There is no simple answer to the problem of classifying cohesive materials at the soil–rock boundary. Various classifications that distinguish geomaterials on the basis of compressive strength of unweathered rock material are summarized in Figure 18, which includes a proposed classification by Kulhawy et al. (1991) in which rock strength is defined relative to that of concrete used in construction, which is assumed to range from 20 kN/m^2 (3 ksi) to 100 kN/m^2 (15 ksi). Rock at the high end of the strength scale (>100 kN/m^2) is classified as strong and in most cases would be expected to be an excellent founding material, except that it would be expensive to excavate. Rock with compressive strength falling within the range of concrete strength is classified as medium and the rock mass could be either weaker or stronger than concrete, depending on weathering and structural features. For rock classified as weak (<20 kN/m^2) foundation capacity is expected to be governed by the strength of the rock mass. Materials defined as cohesive IGMs by O'Neill et al. (1996) fall into this strength range. To account properly for the behavior of weak rock in engineered construction, the following additional factors must be considered carefully: geologic origin, in situ weathering profile, state of stress, groundwater, and construction practices.

A defining characteristic of geomaterials at the soil–rock boundary may be whether or not the in situ material was at one time rock (geologic origin). This is probably the distinguishing feature between clay and rock in Figure 17. The next geologic consideration is the in situ weathering profile. Igneous, sedimentary, or metamorphic rocks subjected to in-place weathering result in geologic profiles that may exhibit the full range of characteristics, for example, as described in Figure 11 (Key), Sheet 2, under "Rock Weathering—Alteration." The descriptive terms are based on recommendations for describing degree of weathering and alteration by the ISRM. One of the criteria for distinguishing between residual soil and completely weathered or altered rock is whether the original rock

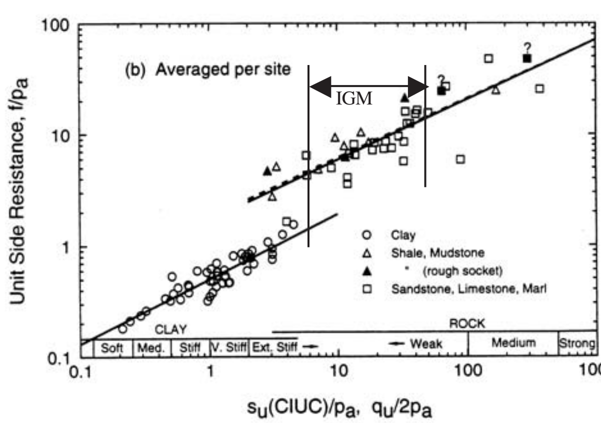


FIGURE 17 Side resistance versus geomaterial strength (Kulhawy and Phoon 1993).

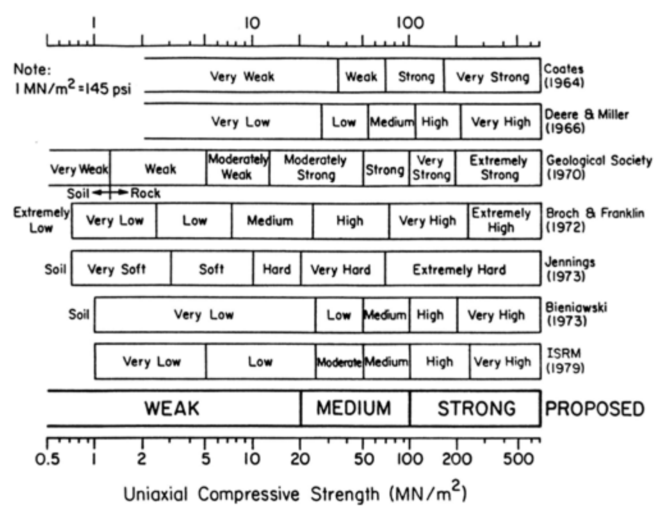


FIGURE 18 Classification for unweathered rock material strength (Kulhawy et al. 1991).

fabric is still apparent. The highest degree of weathering applies to materials derived from rock but for which the rock fabric is not apparent. In this case, the material behavior is controlled by soil fabric and the material should be classified as residual soil, even though it may contain fragments of weathered rock. Materials in which the original minerals have been completely decomposed to secondary minerals but where the original fabric is intact may exhibit rock material behavior governed by rock mass features, including both rock material and discontinuities. The material should be considered to be rock mass, even though it may be highly weathered or altered and exhibit low compressive strength. Judgment is always required in assessing whether material behavior is governed by soil fabric or by rock mass fabric; however, this is a key factor to be assessed in a design approach. Whether a geomaterial is assigned the term "IGM" or "weak rock" is not as important as understanding the geologic processes that give the material its characteristics and engineering properties.

SUMMARY

In this chapter, site characterization methods used to define subsurface conditions at bridge sites underlain by rock were reviewed. The survey shows that eight states currently use geophysical methods to determine depth to bedrock and that seismic refraction is the method used. The literature review suggests that resistivity methods based on the use of multiple arrays can provide detailed profiles that may be useful for both design and construction. Karstic areas in limestone or dolomite terranes with irregular, pinnacled rock surfaces or solution cavities are examples of sites where recent developments in geophysical methods could be applied.

Every agency responding to the survey uses rock core drilling as the primary method of subsurface investigation for rock sockets. Current practice for description and classification of rock core is reviewed. The survey shows that most states routinely determine the RQD of rock core and that the uniaxial compressive strength of intact rock (q_u) is also measured by one of the standardized methods. Also from the survey, it was determined that five states currently classify all rock mass according to the Geomechanics Classification System, in which rock mass is assigned a RMR. Twelve states use RMR occasionally, whereas 14 states indicated that RMR is never used. Some of the analytical methods developed in recent years and described in subsequent chapters of this report require the rock mass classification in terms of RMR. Specifically, RMR can be used to evaluate strength parameters according to the Hoek–Brown failure criterion, a useful approach to quantifying strength of intact or highly fractured rock masses. RMR is used to establish GSI, which is required to use the most up-to-date version of the Hoek–Brown criterion. RMR and/or GSI are useful for estimating rock mass modulus using the empirical correlations given in Table 12. The RMR is also recommended in current FHWA manuals on site characterization and evaluation of soil and rock properties. Wider use of RMR or GSI classification of rock mass is one way that state DOT agencies can use the most up-to-date methods for characterizing RMS and deformation properties.

In situ testing methods that provide information on rock mass modulus include PMT and borehole jack. Five states reported using these tests to obtain modulus values for rock-socket design. To use the best available analytical models for axial and lateral loading, as well as for effective interpretation of load test results, rock mass modulus is a required parameter. Currently, it is noted that there is no definitive in situ method or empirical equation for rock mass modulus that has been calibrated specifically for application to design of rock sockets. A case history example is presented in this chapter illustrating the beneficial use of both in situ testing (borehole jack) and empirical correlations with GSI to establish representative values of rock mass modulus for foundation design.

Site and geomaterial characterization are interrelated with design, construction, and load testing of drilled shafts in rock. For design, Figure 3 shows that rock mass engineering properties required for analysis of rock-socket capacity and load-deformation response are obtained through field and laboratory testing. Table 15 is a summary of rock mass characteristics used in design methods for axial and lateral loading. A large **X** indicates the property is used directly in design equations that are currently applied widely in practice, whereas a small **x** indicates that the characteristic is used indirectly in the design or that it is required for a proposed design method that is not used widely. For example, intact rock modulus E_R is not used directly to analyze load-displacement response of socketed shafts, but may be used to estimate the rock mass modulus E_M, which is used directly in the analytical equations.

Information obtained through the site investigation process will be used not only by design engineers but by contractors who will bid on the work and construct the foundations. As indicated in the flowchart shown in Figure 3, a goal of site characterization is to obtain information on constructability. O'Neill and Reese (1999) point out that contractors will be most interested in knowing the difficulties that might be encountered in drilling the rock. Specific information that is useful in assessing the difficulty of drilling in rock includes loss or gain of drill water; rock type with lithological description; rock strength; characteristics of weathering; and rock mass characteristics such as the presence, attitude, and thickness of bedding planes, foliation, joints, faults, stress cracks, cavities, shear planes, or other discontinuities. Boring logs, containing most of the information determined by the site investigation, are incorporated directly into the construction plans by most state DOTs. Any of the above information not given in the boring logs should be made available to bidders to facilitate informed decisions. The same information will be used by the design engineer to forecast potential construction methods and construction

TABLE 15
ROCK MASS ENGINEERING PROPERTIES REQUIRED FOR ROCK-SOCKET DESIGN

Rock Mass Characteristic	Design Applications: Axial Loading: Unit Side Resistance	Axial Loading: Unit Base Resistance	Axial Loading: Axial Load-Displacement	Lateral Loading: Ultimate Resistance	Lateral Loading: Load-Displacement: Continuum Methods	Lateral Loading: Load-Displacement: *p-y* Curve Parameter
Compressive strength, intact rock, q_u	**X**	**X**		**X**	x	**X**
Split tensile strength, intact rock, q_t	**X**					
Rock mass strength by Mohr–Coulomb or Hoek and Brown		**X**		x		**X**
Shear strength of joint surfaces		**X**		x		
Elastic modulus, intact rock, E_R	x		x		x	x
Elastic modulus, rock mass, E_M	x		**X**		**X**	**X**
Rock quality designation (RQD)	x	**X**	x		x	
Rock Mass Rating (RMR)		**X**	x		x	
Geological Strength Index (GSI)		**X**	x		x	**X**

Notes: **X** = property is used directly in equations that are currently applied widely in practice.
x = characteristic is used indirectly in the design or it is required for a proposed design method not widely used.

problems to develop specifications for the project and to make cost estimates. Rock cores should be photographed and, when practical, retained for examination by prospective bidders.

Field load testing, shown in the flowchart of Figure 3 and described in chapter five, provides direct verification of design assumptions regarding axial and lateral capacity and load-deformation response. Results of field load testing also provide the basis for many of the design methods discussed in the next two chapters. For correct interpretation of load test results, it is imperative that subsurface conditions and soil–rock engineering properties be evaluated as carefully as possible. The properties required for design and listed in Table 15 are also required for load test interpretation and for proper extrapolation of load test results to design.

CHAPTER THREE

DESIGN FOR AXIAL LOADING

SCOPE

A rock-socketed drilled shaft foundation must be designed so that the factored axial resistance is not less than the effects of the factored axial loads. At the strength limit state, side and base resistances of the socketed shaft are taken into account. Design for the service limit state accounts for tolerable movements of the structure and requires analysis of the axial load-deformation response of the shaft. In this chapter, current understanding of rock socket response to axial loading is summarized, based on a literature review. Analysis methods for predicting axial load capacity and axial load-displacement response of shafts in rock and IGM are then reviewed and evaluated for their applicability to highway bridge practice.

RELATIONSHIP TO GEOMATERIAL CHARACTERIZATION

Design for axial loading requires reliable site and geomaterial characterization. Accurate geometric information, especially depth to rock and thickness of weathered and unweathered rock layers, is essential for correct analysis of axial resistance. This information is determined using the tools and methods outlined in the previous chapter, principally core drilling supplemented by geophysical methods. Rock mass characterization using the Geomechanics System (Bieniawski 1989) provides a general framework for assessing the overall quality of the rock mass and its suitability as a foundation material. Engineering properties of the intact rock and the rock mass are used directly in the analysis methods described in this chapter. For example, empirical relationships have been derived between rock-socket unit-side resistance and uniaxial strength of intact rock. Base capacity, analyzed as a bearing capacity problem, may require uniaxial compressive strength of intact rock, shear strength of discontinuities, or the Hoek–Brown strength parameters of fractured rock mass, depending upon the occurrence, orientation, and condition of joint surfaces in the rock mass below the base. For analysis of axial load-displacement response, the rock mass modulus is required. Modulus may be determined from in situ testing, such as pressuremeter or borehole jack tests, or estimated from rock mass classification parameters as summarized in Table 12. Engineering properties of rock mass used in conjunction with LRFD methods should be mean values, not minimum values sometimes used in geotechnical practice.

Several methods proposed in recent years for analysis of both axial and lateral load response of rock sockets require, as an input parameter, the GSI proposed by Hoek et al. (1995, 2002). GSI is also correlated to the parameters that establish the Hoek–Brown strength criterion for fractured rock masses. Although GSI is not widely used in foundation engineering practice at the present time, it likely will become a standard rock mass characteristic for rock-socket design.

LOAD TRANSFER BEHAVIOR OF ROCK SOCKETS

Compression Loading

A compressive force applied to the top (head) of a rock-socketed drilled shaft is transferred to the ground through (1) shearing stress that develops at the concrete–rock interface along the sides of the shaft and (2) the compressive normal stress that develops at the horizontal interface between the base of the shaft and the underlying rock. A conceptual model of the load transfer can be illustrated by considering a generalized axial load versus displacement curve as shown in Figure 19 (Carter and Kulhawy 1988). Upon initial loading, shearing stress develops along the vertical shaft–rock interface. For a relatively small load, displacement is small and the stress–strain behavior at the shaft–rock interfaces is linear (line OA). There is no relative displacement ("slip") between the concrete shaft and surrounding rock and the system may be modeled as being linearly elastic. With increasing load, the shear strength along some portion of the shaft sidewall is exceeded, initiating rupture of the "bond" and relative slip at the shaft–rock interface. The load-displacement curve becomes nonlinear as rupture, and slip progress and a greater proportion of the applied load is transferred to the base (line AB). At some point, the full side resistance is mobilized, and there is slip along the entire surface ("full slip" condition), and a greater proportion of the applied load is transferred to the shaft base (beyond point B in Figure 19). If loading is continued to a displacement sufficient to cause failure of the rock mass beneath the base, a peak compressive load may be reached. In practice, design of drilled shafts in rock requires consideration of (1) deformation limits and (2) geotechnical and structural capacity (strength limit states). Geotechnical capacity in compression is evaluated in terms of limiting side and base resistances. Load transfer in uplift involves the same mechanisms of side resistance mobilization as described previously for compression.

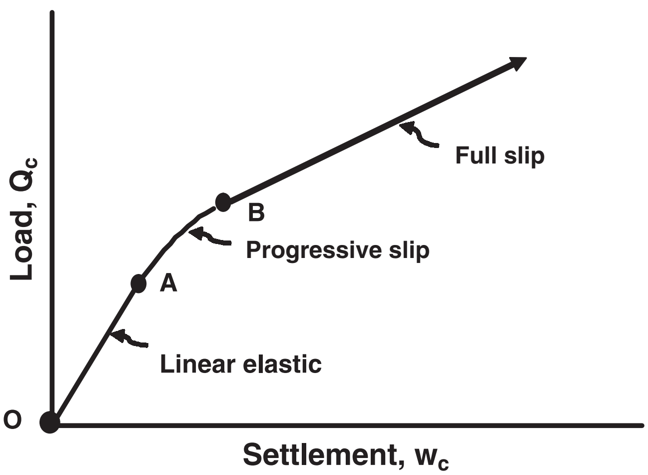


FIGURE 19 Idealized load-displacement behavior.

A rigorous model for the behavior of a rock-socketed drilled shaft under axial compression would provide a prediction of the complete load-displacement curve. In reality, the mechanisms of side and base load transfer are complex and can only be modeled accurately through the use of sophisticated numerical methods, such as finite-element or boundary-element methods. Input parameters required for accurate modeling are not normally available for design. In recent years, several researchers have presented simplified methods of analysis that provide bounds on the expected and observed behaviors for shafts that fall within the range of conditions typically encountered in practice. Methods most relevant to rock-socketed bridge foundations are presented in this chapter. Some of the more important behavioral aspects pertaining to side and base resistance and their mobilization are described first.

Side Resistance Mechanisms

The conditions of the sidewall interface determine the strength and load transfer in side resistance. Side resistance often exhibits a "bond" component that may exist physically as a result of the cementation between the concrete and rock and from mechanical interlocking between asperities along the interface. If the shearing strength of the interface is modeled as a Mohr–Coulomb material, the bond component can be considered as the interface adhesion, c. If displacements are sufficient, the interface bond is ruptured and the cohesion component of resistance may be diminished. The second mechanism of resistance is frictional. Physically, the frictional resistance can have two components. The first is the sliding friction angle of the interface, ϕ. The second is mechanical dilatancy, which can be described as an increase in the interface normal stress in response to the normal displacement (dilation) required to accommodate shear displacement of a rough surface. For mathematical simplicity, dilatancy can be quantified in terms of the angle of dilation (ψ), where ψ corresponds to the average angle of triangular asperities from the direction of shear displacement. The interface shear strength (τ) is then given by

$$\tau = c + \sigma_n \tan(\phi + \psi) \tag{18}$$

in which σ_n = interface normal stress. Physically, all three components of strength (c, ϕ, ψ) may vary with displacement. The initial shear strength may have both cohesive and frictional components. Following rupture, the cohesion is probably decreased and dilation is mobilized. With further displacement, dilation may cease and resistance may be purely frictional and correspond to the residual friction angle. In addition, field conditions of construction can significantly affect the nature of the sidewall interface and, in practice, will determine the relative contributions of cohesion, friction, and dilatancy to shearing resistance. For example, the bond (adhesion) may be partially or completely prevented by the presence of drilling slurry, or by "smearing," which occurs in some argillaceous rocks or in rocks that are sensitive to property changes in the presence of water. Dilatancy is a function of interface roughness and shear strength of the intact rock forming the asperities. Sidewall roughness is determined in part by rock type and texture, but can also be affected by construction tools and practices. Practices that result in a "smooth" sidewall will reduce dilatancy compared with practices that provide a "rough" sidewall (Williams and Pells 1981; Horvath et al. 1983).

Johnston and Lam (1989) made detailed investigations of the rock–concrete interface with the goal of better understanding the factors that determine interface roughness and its influence on side-load transfer. Figure 20*a* shows an idealized section of a rock socket following construction. An initial normal force exists between the rock and concrete. When the shaft is loaded vertically, the shearing resistance develops and the rock mass will deform elastically until slip occurs. Figure 20*a* and *b* show the positions of the shaft before and after slip displacement. These two conditions are represented by 2-D models in Figure 20*c* and *d*, respectively. Figure 20*d* illustrates the dilation that occurs as a result of geometrical constraints. Dilation occurs against the surrounding rock mass, which must deform to compensate for the increase in socket diameter, resulting in an increase in the interface normal stress. The average normal stress increase ($\Delta\sigma_n$) can be approximated using the theoretical solution that describes expansion of an infinite cylindrical cavity, as follows:

$$\Delta\sigma_n = \frac{E_M}{1+\nu}\frac{\Delta r}{r} \tag{19}$$

where E_M and ν are the rock mass modulus and Poisson's ratio, respectively; Δr is the dilation, and r is the original shaft radius. A normal stiffness K can be defined as the ratio of normal stress increase to dilation, as follows:

$$K = \frac{\Delta\sigma_n}{\Delta r} = \frac{E_M}{r(1+\nu)} \tag{20}$$

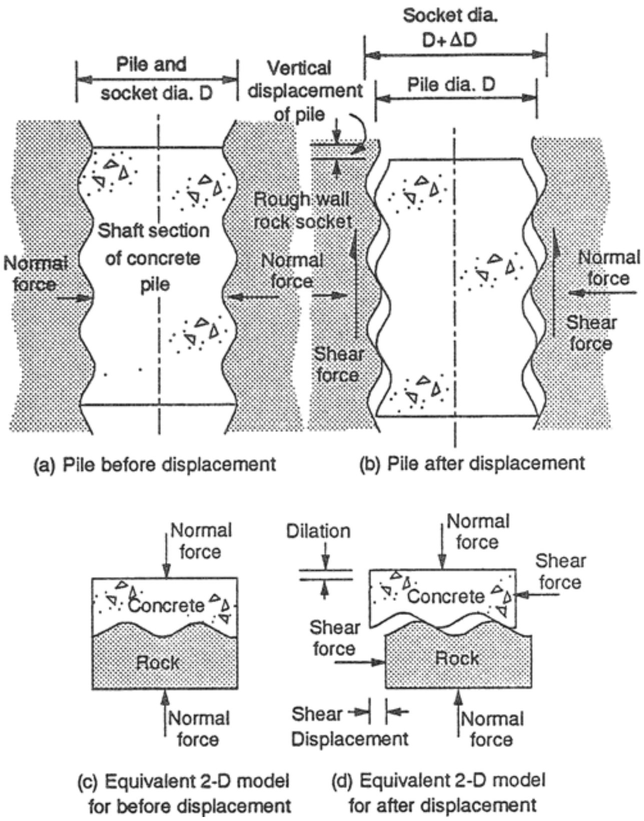


FIGURE 20 Idealized rock–concrete interface under axial loading (Johnston and Lam 1989).

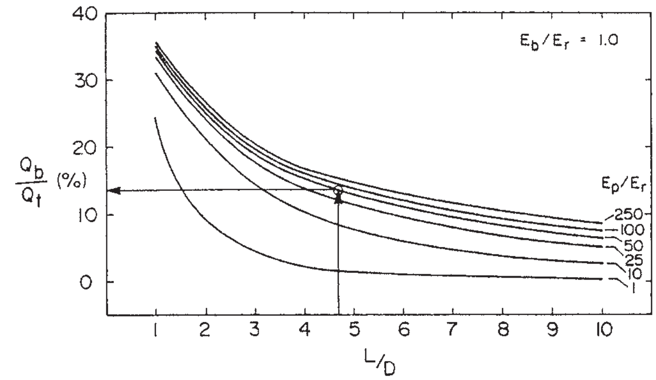


FIGURE 21 Theoretical base load transfer (Rowe and Armitage 1987b).

Assuming the deformation Δr is small compared with r, and E_M and v can be considered to be constant for the stress range considered, it follows that the behavior of the rock–concrete interface is governed by CNS conditions. This concept forms the basis of the CNS direct shear test, in which the normal force is applied through a spring (Johnston et al. 1987; Ooi and Carter 1987).

Shaft Geometry and Relative Rigidity

Load transfer in a rock socket depends on the geometry, expressed by the embedment ratio (depth/diameter), and the stiffness of the concrete shaft relative to stiffness of the surrounding rock mass. Figure 21, based on finite-element analysis, illustrates this behavior for the initial (no slip) part of the load-displacement curve. In Figure 21, L = socket length, D = shaft diameter, E_p = modulus of the shaft, E_r = modulus of rock mass above the base, E_b = modulus of rock mass below the base, Q_b = load transmitted to the base, and Q_t = load applied to the head of the shaft. The portion of applied axial compressive load that is transferred to the base is shown as a function of embedment ratio and modulus ratio. With increasing embedment ratio, the relative base load transfer decreases. For embedment ratios of 10, less than 10% of the applied load is transferred to the base. The effect of modulus ratio is more significant at lower embedment ratios and, in general, base load transfer increases with increasing modulus ratio. Cases that result in the most base load transfer correspond to low embedment ratio with high modulus ratio (shaft is rigid compared to rock mass); whereas the smallest base load transfer occurs at higher embedment ratios and low modulus ratio (stiff rock mass).

The proportion of load transferred to the base will also vary with the stiffness of the rock mass beneath the base of the shaft relative to the stiffness of the rock along the side. In many situations, a rock socket is constructed so that the base elevation corresponds to relatively "sound" or "intact" rock, and it may be necessary to excavate through weathered or fractured rock to reach the base elevation. In that case, the modulus of the rock mass below the base may be greater than that of the rock along the sidewall of the socket. Osterberg and Gill (1973) demonstrate the difference in load transfer in side and base resistances for two conditions, one in which the base modulus is twice that of the sidewall rock modulus and one where the base rock has a much lower modulus than that of the rock surrounding the shaft side. Their results show that base load transfer increases as the ratio E_b/E_r increases (Figure 22).

Load transfer is affected significantly by the roughness of the sidewall interface. Fundamentally, this can be explained by the higher load transfer in side shear reducing the proportion of load transferred to the base. Because side resistance increases with interface roughness, rock sockets with higher interface roughness will transfer a higher proportion of load in side resistance than smooth sockets. The complex interrelationships between load transfer, interface roughness, modulus ratio, and embedment ratio have been studied by several researchers, and the reader is referred to Pells et al. (1980), Williams et al. (1980), Rowe and Armitage (1987a), and Seidel and Collingwood (2001) for more detailed discussions. Six state DOTs indicated the use of grooving tools or other methods to artificially roughen the sidewalls of rock-socketed shafts.

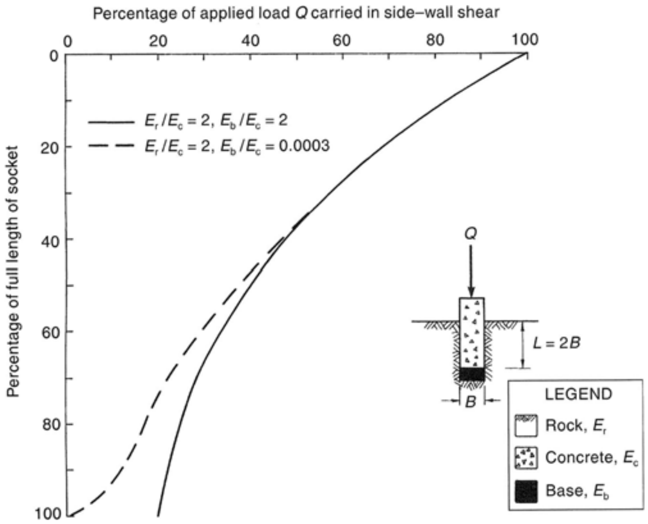


FIGURE 22 Effect of rock mass modulus at base on axial load transfer (Wyllie 1999, based on Osterberg and Gill 1973).

Base Condition

In many cases encountered in practice there is uncertainty about conditions at the base of the shaft. Most transportation agencies include, in their drilled shaft specifications, limits on the amount of drill cuttings, water, or slurry that is permissible at the base before concrete placement (Survey Question 34). However, compliance is not always verified and in some cases there is a perception that it is not practical to clean or inspect the base of the socket. In these cases, the designer may assume that base resistance will not develop without large downward displacement and for this reason base resistance is sometimes neglected for design purposes. Ten states indicated in their responses to Question 14 of the survey that rock-socketed shafts are sometimes designed under the assumption of side resistance only. The draft Interim 2006 AASHTO *LRFD Bridge Design Specifications* state that "Design based on side-wall shear alone should be considered for cases in which the drilled hole cannot be cleaned and inspected or where it is determined that large movements of the shaft would be required to mobilize resistance in end bearing." Table 16 lists the most common reasons cited by foundation designers for neglecting base resistance in design, along with actions that can be taken to address the concern.

Crapps and Schmertmann (2002) suggest that accounting for base resistance in design and using appropriate construction and inspection techniques to ensure quality base conditions is a better approach than neglecting base resistance. The authors support their recommendations with field load test results in which load transferred to the base was measured. The database consisted of 50 Osterberg load cell (O-cell) tests and 22 compression tests in which the load was applied to the top of the shaft. Of those, 30 of the O-cell tests and 4 of the top load tests were conducted on rock-socketed shafts. Eight of the O-cell tests (27%) showed evidence of bottom disturbance in the O-cell load-displacement curves. Results from the 34 tests are plotted in Figure 23 in terms of base load ratio (Q_b = base load, Q_t = actual top load or top load inferred from the O-cell test) versus socket-effective depth-to-diameter ratio (L/B). For some of the shafts, multiple measurements are included at different values of load and displacement. However, all of the base load ratio values correspond to downward displacements at the top of the shaft that range from 2.5 mm to 25.4 mm, with most in the range of from 3 to 15 mm. These values are within the service limit state for most bridge foundations. Additional details regarding the test shafts, subsurface profiles, and load test interpretation are given in Crapps and Schmertmann (2002).

Several important observations arise from the data shown in Figure 23. First, base resistance mobilization represents a significant contribution to overall shaft resistance at downward displacements corresponding to typical service loads. Second, the magnitude of base resistance is generally greater than predicted by elasticity-based numerical solutions (e.g., compare with Figure 21). The dashed lines in Figure 23 represent approximate upper and lower bounds to the data from top load tests and O-cell tests without bottom disturbance. For the most part, O-cell tests that exhibited bottom disturbance fall below the lower-bound curve. Although the data are not sufficient to provide design values of base load transfer in advance for a given situation, they provide compelling evidence that shaft design in rock should account properly for base resistance, and that quality construction and inspection aimed at minimizing base disturbance can provide performance benefits.

Time Dependency

Time-dependent changes in load transfer may occur in rock-socketed shafts under service load conditions. Ladanyi (1977) reported a case in which the bearing stress at the base of an instrumented rock socket increased, at a steadily decreasing rate, over a period of 4 years; although the total applied head

TABLE 16
REASONS FOR NEGLECTING BASE RESISTANCE AND CORRECTIVE ACTIONS (after Crapps and Schmertmann 2002)

Reason Cited for Neglecting Base Resistance	Correction
Settled slurry suspension	Utilize available construction and inspection methods
Reluctance to inspect bottom	Utilize available construction and inspection methods
Concern for underlying cavities	Additional inspection below base
Unknown or uncertain base resistance	Load testing

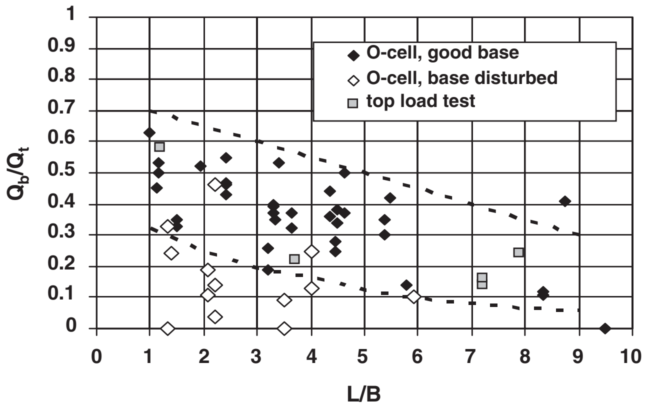


FIGURE 23 Base load transfer interpreted from load tests (data from Crapps and Schmertmann 2002).

load remained essentially constant. However, the percentage of total load carried by the base after this period was still less than 10% of the applied load and agreed quite well with predictions based on elastic theory. Tang et al. (1994) described a similar monitoring program on a shaft socketed into karstic dolomite supporting a building on the University of Tennessee campus. Some change in load transfer occurred following the end of construction; however, most of the change was from side resistance in the overlying soil (decreased) to side resistance in the rock socket (increased). Neither case would suggest changes in design of rock sockets to account for the time dependency of load transfer mechanisms.

CAPACITY UNDER AXIAL LOADING

The factored axial resistance of a drilled shaft in compression is the sum of the factored side resistance and the factored base resistance. The factored resistances are calculated by multiplying appropriate resistance factors by the nominal resistances, which are generally taken as the ultimate values. One approach to the design process depicted in Figure 3 of chapter one is to size the foundation initially to achieve a factored resistance that exceeds the factored loads. The trial design is then analyzed to predict load-displacement response. If necessary, revised trial dimensions can then be analyzed until all of the design criteria are satisfied, including the movement criteria associated with the service limit state. In the case of axial loading, the ultimate side and base resistances are required to establish the initial trial design.

Side Resistance

The ultimate side resistance of a rock socket is the summation of peak shearing stress acting over the surface of the socket, expressed mathematically by

$$Q_s = \int_{\text{surface}} f_{su}\, dA = \pi B \int_L f dz \qquad (21)$$

in which Q_s = total side resistance (force), f_{su} = unit side resistance (stress), A = surface area along the side of the socket, B = socket diameter, and L = socket length. In practice, socket side resistance capacity is calculated by assuming that a single average value of unit side resistance acts along the concrete–rock interface, for each rock layer. This value of f is multiplied by the area of the interface to obtain total side resistance Q_s, or

$$Q_s = f_{su} \times \pi B L \qquad (22)$$

Methods for predicting socket side resistance are, therefore, focused on the parameter f_{su}.

The interaction between a rock mass and drilled shaft that determines side resistance is complex. The principal factors controlling this interaction include:

- Rock material strength;
- Rock mass structure (discontinuities);
- Modulus of the concrete relative to modulus of the rock mass;
- Shear strength mobilized by dilatancy;
- Confining stress; and
- Construction-related factors, including roughness of shaft–rock interface.

Geomechanical models that account for these factors (to varying degrees) are described in the literature (e.g., Rowe and Pells 1980); however, the methods required to obtain the necessary input parameters normally fall outside the scope of a typical investigation conducted for the design of highway bridge foundations. More realistically, methods based on the strength of intact rock, in some cases with modifications to account for one or more of the other factors, have been used successfully and are more rational than some of the strictly empirical methods or presumptive values. This approach represents a practical compromise between oversimplified empirical methods and more sophisticated numerical methods that might be warranted only on larger projects. The methods are summarized here.

Methods Based on Rock Compressive Strength

A practical approach to evaluating average unit side resistance is to relate f_{su} to the strength of the intact rock material. The rock material strength parameter most often measured is the uniaxial compressive strength (q_u). In this approach, values of f_{su} are determined from full-scale field load tests in which ultimate side resistance (Q_s) has been determined. This value is divided by the socket side area (A_s) to obtain an average value of unit side resistance at failure:

$$f_{su} = Q_s/A_s \qquad (23)$$

Early studies relating f_{su} to q_u include those by Rosenberg and Journeaux (1976) and Horvath (1978, 1982). Other researchers have continued to expand the available database and propose equations relating unit side resistance to rock strength on the basis of statistical best-fit analyses. Notable studies include those of Williams and Pells (1981), Rowe and Armitage (1984, 1987b), Bloomquist and Townsend (1991), McVay et al. (1992), and Kulhawy and Phoon (1993). These studies, including the proposed equations relating unit side resistance to rock strength are reviewed briefly.

Horvath and Kenney (1979) proposed the following correlation between side resistance and compressive strength:

$$f_{su} = b\sqrt{q_u} \tag{24}$$

in which f_{su} = ultimate unit side resistance, q_u = compressive strength of the weaker material (rock or concrete), and where b ranges from 0.2 (smooth) to 0.3 (rough). Both f_{su} and q_u in Eq. 24 are in units of MPa. Eq. 24 can be expressed in normalized form by dividing both unit side resistance and compressive strength by atmospheric pressure ($p_a = 0.1013$ MPa). This results in the following expression, which is equivalent to Eq. 24 with $b = 0.2$:

$$\frac{f_{su}}{p_a} = 0.65\sqrt{\frac{q_u}{p_a}} \tag{25}$$

A modified relationship was recommended by Horvath et al. (1983) to account for shafts with artificially roughened (grooved) sockets. The suggested relationship is given by

$$f_{su} = 0.8[RF]^{0.45} \times q_u \tag{26}$$

$$RF = \frac{\Delta r_h}{r_s}\frac{L_t}{L_s} \tag{27}$$

in which RF = roughness factor, Δr_h = average height of asperities, r_s = nominal socket radius, L_s = nominal socket length, and L_t = total travel distance along the socket wall profile. A device (caliper) was used to measure field roughness for determination of the parameters needed in Eq. 27, as described by Horvath et al. (1993).

The FHWA *Drilled Shaft Manual* (O'Neill and Reese 1999) and the draft 2006 Interim AASHTO *LRFD Bridge Design Specifications* have adopted Eqs. 24 to 27 as a recommended method for selection of design side resistance for shafts in rock. AASHTO refers to this as the Horvath & Kenney method. A socket that is not specified to be artificially roughened by grooving is considered "smooth" and side resistance is governed by Eq. 25. If the socket is artificially roughened, Eq. 26 is recommended; however, this requires estimation or measurement of roughness as defined by Eq. 27.

Rowe and Armitage (1987b) summarized the available data on side resistance of rock sockets, including the databases used by Williams et al. (1980), Williams and Pells (1981), and Horvath (1982). The suggested correlation for regular clean sockets, defined as roughness classes R1, R2, and R3 in Table 17, is given as

$$f_{su} = 0.45\sqrt{q_u} \tag{28}$$

To account for rough sockets, defined as category R4, side resistance is increased and the following is recommended:

$$f_{su} = 0.6\sqrt{q_u} \tag{29}$$

The correlation suggested by Horvath and Kenney (1979) as given by Eq. 25 represents a lower bound to the data used by Rowe and Armitage (1987*b*)

Kulhawy and Phoon (1993) incorporated the database compiled by Rowe and Armitage, which included more than 80 load tests from more than 20 sites, and the data reported by Bloomquist and Townsend (1991) and McVay et al. (1992) consisting of 47 load tests to failure from 23 different Florida limestone sites. Linear regression was conducted on two sets of data, one consisting of all data points and the other using data that were averaged on a per-site basis. Averaging eliminates the bias associated with multiple load tests conducted at many of the sites. All stresses are normalized by atmospheric pressure, p_a (0.1013 MN/m^2) and normalized values of one-half of the uniaxial compressive strength were plotted against normalized values of average unit side resistance on a log–log plot as shown in Figure 24 (shown previously as Figure 17).

The following exponential expression provides a best-fit to the available data for rock:

$$\frac{f_{su}}{p_a} = C \times \sqrt{\frac{q_u}{2p_a}} \tag{30}$$

TABLE 17
SHAFT ROUGHNESS CLASSIFICATION (after Pells et al. 1980)

Roughness Class	Description
R1	Straight, smooth-sided socket; grooves or indentations less than 1 mm deep
R2	Grooves 1–4 mm deep, >2 mm wide, spacing 50–200 mm
R3	Grooves 4–10 mm deep, >5 mm wide, spacing 50–200 mm
R4	Grooves or undulations >10 mm deep, >10 mm wide, spacing 50–200 mm

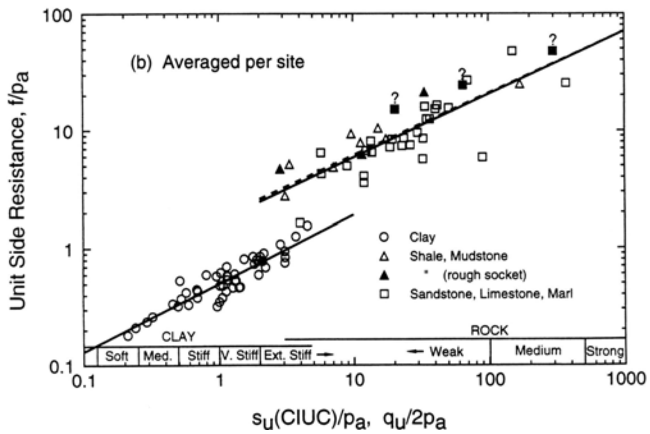


FIGURE 24 Unit side resistance versus strength (Kulhawy and Phoon 1993).

where values of the coefficient $C = 1$ represents a reasonable lower bound, $C = 2$ represents the mean behavior, and $C = 3$ corresponds to an upper bound for artificially roughened sockets. Kulhawy and Phoon (1993) noted that the expressions and variations are consistent with those reported by Rowe and Armitage (1987b). Values of f_{su} obtained using Eq. 30 with $C = 2$ (mean) are identical to the values obtained using Eq. 28 (Rowe and Armitage) corresponding to the mean trend for smooth sockets. The expression of Rowe and Armitage for rough sockets (Eq. 29) corresponds to Eq. 30 with $C = 2.7$.

Kulhawy et al. (2005) recently reexamined the data available and attempted to evaluate them in a more consistent manner. Only data showing load-displacement curves to failure were incorporated into the analysis, so that the "interpreted failure load" could be established in a consistent manner. Based on the updated analysis, the authors recommend the use of Eq. 30 with $C = 1$ for predicting side resistance of normal rock sockets for drilled shafts. The authors also note the importance of using compressive strength values (q_u) obtained from laboratory uniaxial compression tests, not from point load tests.

In summary, Eq. 30 with $C = 1$ provides a conservative estimate of design ultimate side resistance, based on the most up-to-date analysis of the available data. Use of C values greater than 1 for design should be verified by previous experience or load testing. Load test results that exhibit values of C in the range of 2.7 to 3 demonstrate the potential increase in side resistance that is possible if the sidewalls of the socket are roughened. These upper-bound values of f_{su} should only be considered when they can be validated by field load testing.

The methods described rely on empirical relationships between side resistance and a single parameter, uniaxial compressive strength, to represent the rock mass. There is significant scatter in the database because the relationships do not capture all of the mechanisms affecting unit side resistance and because the database incorporates load test results for many different rock types. Use of these empirical correlations for design should therefore be conservative (i.e., $C = 1$) unless a site-specific correlation has been developed that justifies higher values. Research on development of methods that account for these additional factors affecting peak side resistance is summarized here.

Methods Based on Additional Rock Mass Parameters

Williams et al. (1980) and Rowe and Armitage (1987a) point out that unit side resistance determined strictly by empirical correlations with uniaxial compressive strength does not account explicitly for the degree of jointing in the rock mass. Figure 25 shows the potential influence on average side resistance (denoted by τ in the figure) of jointing in terms of the ratio of rock mass modulus to modulus of intact rock. Both theoretical and experimental curves show substantially reduced side resistance for rock that may have high intact strength and stiffness but low mass modulus.

The draft Interim 2006 AASHTO *LRFD Bridge Design Specifications* adopt the Horvath and Kenney method described earlier (Eqs. 25–27), with the following modification as recommended O'Neill and Reese (1999). Values of unit side resistance calculated by either Eq. 25 or Eq. 26 are modified to account for rock mass behavior in terms of RQD, modulus ratio (E_M/E_R), and joint condition using the factor α as defined in Table 18. In Table 18, f_{des} is the reduced unit

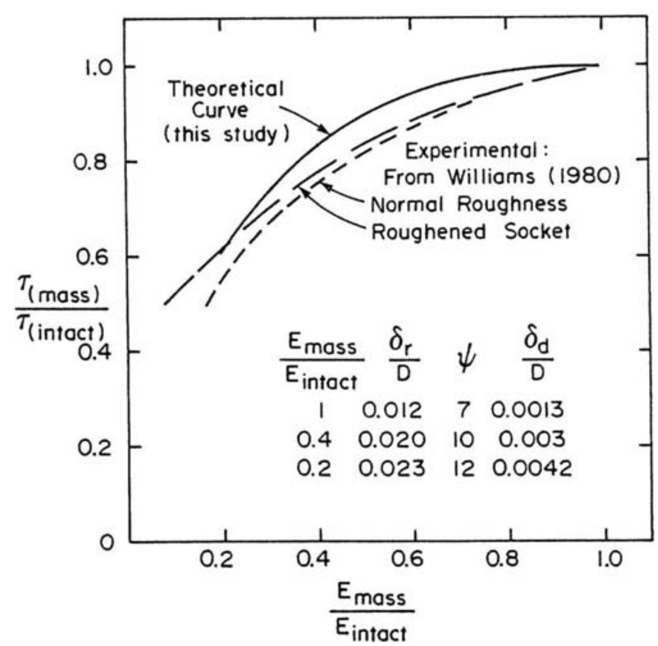


FIGURE 25 Effect of rock mass modulus on average unit side resistance (Rowe and Armitage 1987a).

TABLE 18
SIDE RESISTANCE REDUCTION BASED ON MODULUS REDUCTION
(O'Neill and Reese 1999)

E_M/E_R	$\alpha = f_{des}/f_{su}$
1.0	1.0
0.5	0.8
0.3	0.7
0.1	0.55
0.05	0.45

side resistance recommended for design. The modulus reduction ratio (E_M/E_R) is given in Table 13, in chapter two, based on RQD. However, application of the α-factor may be questionable because the RQD and rock mass modulus were not accounted for explicitly in the original correlation analysis by Horvath and Kenney (1979). Because the load test data included sites with RQD less than 100 and modulus ratio values less than one, it would appear that these factors affected the load test results and are therefore already incorporated into the resulting correlation equations.

Interface roughness is identified by all researchers as having a significant effect on peak side resistance. Pells et al. (1980) proposed the roughness classification that assigns a rock socket to one of the categories R1 through R4 as defined in Table 17. The criteria are based on observations of sockets drilled in Sydney sandstone and the classification reportedly forms the basis for current practice in that city (Seidel and Collingwood 2001). The Rowe and Armitage (1987b) correlation equations were developed by distinguishing between roughness classes R1–R3 (Eq. 28) and roughness class R4 (Eq. 29). Horvath et al. (1983) proposed the roughness factor (*RF*) defined in Eqs. 26 and 27 as presented earlier. Despite these efforts, selection of side resistance for rock-socket design in the United States is done mostly without considering interface roughness explicitly.

Seidel and Haberfield (1994) developed a theoretical model of interface roughness that accounts for the behavior and characteristics of socket interfaces under CNS conditions. Roughness is modeled using a quasi-probabilistic approach that involves fractal geometry to predict the distribution and characteristics of asperities. Results of the interface model and laboratory CNS testing are incorporated into the computer program ROCKET that predicts the axial load-displacement curve, including post-peak behavior. Extending this work, Seidel and Collingwood (2001) proposed a nondimensional parameter defined as the shaft resistance coefficient (SRC) to account for the factors that influence side resistance, as follows:

$$\mathrm{SRC} = \eta_c \frac{n}{1+\nu} \frac{\Delta r}{d_s} \tag{31}$$

$$f_{su} = (\mathrm{SRC})\, q_u \tag{32}$$

in which η_c = construction method reduction factor, as defined in Table 19; n = ratio of rock mass modulus to uniaxial compressive strength of intact rock (E_M/q_u); ν = Poisson's ratio; Δr = mean roughness height; and d_s = socket diameter. Implementation of the SRC in design requires an estimate of socket roughness in terms of Δr. As noted by the authors, reliable measurements of roughness are not undertaken in routine design. However, the SRC factor incorporates many of the significant parameters that influence side resistance, including rock mass modulus, Poisson's ratio, and intact rock strength, and provides a framework for taking into account socket roughness and construction effects.

The SRC method represents the type of approach that holds promise for improved methods for selecting design side resistance. Although more detailed guidance is required for determination of socket roughness and construction effects, improvements in reliability of design equations are possible only if the relevant factors controlling side resistance are incorporated properly. Advancement of the SRC or other robust methods can be facilitated by promoting the awareness of engineers involved in field load testing of the importance of collecting appropriate data on rock mass characteristics. Documentation of RMS and modulus along with careful observation and documentation of construction procedures would allow these methods to be evaluated against load test results. The key parameter that is currently missing from the database is socket roughness. O'Neill et al. (1996) point out that roughness can be quantified approximately by making electronic or mechanical caliper logs of the borehole, and that such borehole calipers are available commercially. Seidel and Collingwood (2001) describe a device called the Socket-Pro that is operated remotely and records sidewall roughness to depths of 60 m.

Geomaterial-Specific Correlations

Correlations between unit side resistance and intact rock strength that are based on a global database (e.g., Figure 24, Eq. 30) exhibit scatter and uncertainty because the results reflect the variations in interface shear strength of different rock types, interface roughness, and other factors that control side resistance. For this reason, selection of design side resistance values based on such correlations should be considered as first-order estimates and the philosophy underlying their use for design is that a lower-bound, conservative relationship should be used (e.g., C = 1 in Eq. 30). Alternatively, correlations have been developed for specific geomaterials. Correlations identified by the literature review and the survey are summarized here.

Florida Limestone Limestone formations in Florida are characterized by highly variable strength profiles, the presence of cavities that may be filled with soil, and interbedding of limestone with sand and marine clay layers (Crapps 1986). Locally, geotechnical engineers distinguish between "lime-

TABLE 19
CONSTRUCTION METHOD REDUCTION FACTORS, η_c (Seidel and Collingwood 2001)

Construction Method	η_c
Construction without drilling fluid	
Best practice construction and high level of construction control (e.g., socket sidewalls free of smear and remolded rock)	1.0
Poor construction practice or low-quality construction control (e.g., smear or remolded rock present on rock sidewalls)	0.3–0.9
Construction under bentonite slurry	
Best practice construction and high level of construction control	0.7–0.9
Poor construction practice or low level of construction control	0.3–0.6
Construction under polymer slurry	
Best practice construction and high level of construction control	0.9–1.0
Poor construction practice or low level of construction control	0.8

rock" and "limestone"; the former defined informally as material with q_u less than approximately 13.8 MN/m^2 (2,000 psi). McVay et al. (1992) conducted a study of design methods used to predict unit side resistance of drilled shafts in Florida limestone. Based on a parametric finite-element study and a database of 14 case histories consisting of full-scale load tests and field pullout tests, the following expression was found to provide a reasonable estimate of ultimate unit side resistance:

$$f_{su} = \frac{1}{2}\sqrt{q_u}\sqrt{q_t} \tag{33}$$

In which q_u = uniaxial compressive strength and q_t = split tensile strength. To account for the effect of material strength variability on side resistance, the authors recommend a minimum of 10 (preferably more) core samples be tested in unconfined compression and splitting tensile tests. The mean values of q_u and q_t are used in Eq. 33. The standard error of the mean from the laboratory strength tests can be used to estimate the expected variation from the mean side resistance, for a specified confidence level.

According to Lai (1998), design practice by the Florida DOT is based on a modified version of the McVay et al. relationship in which spatial variations in rock quality are incorporated by multiplying the unit side resistance, according to Eq. 33, by the average percent recovery (REC) of rock core expressed as a decimal, or:

$$(f_{su})_{\text{design}} = \frac{\text{REC(\%)}}{100\%}\left[\frac{1}{2}\sqrt{q_u}\sqrt{q_t}\right] \tag{34}$$

Lai (1998) also recommends using larger diameter double-tube core barrels (61 mm to 101.6 mm inner diameter) for obtaining samples of sufficient quality for laboratory strength tests. Analysis of the laboratory strength data involves discarding all data points above or below one standard deviation about the mean, then using the mean of the remaining values as input to Eq. 34. Crapps (2001) recommends using RQD in place of REC in Eq. 34 and points out that values of q_u and q_t used for design should be limited to the design strength of the shaft concrete.

Side resistance values in Florida limestone have also been evaluated using a small-scale field pullout test devised by Schmertmann (1977) for the Florida DOT and shown schematically in Figure 26. A grout plug is placed into a 140-mm-diameter cored hole at the bottom of a 165-mm-diameter hole drilled to the test depth in rock. Overburden soils are supported by a 200-mm-diameter casing. The grout plug is reinforced with a wire cage and a threaded high-strength steel bar extends from the bottom of the plug to the ground surface. A center hole jack is used to apply a pullout force to the bar. The grout plug is typically 610 mm (2 ft) in length, but other lengths are also used. The average unit side resistance is taken as the measured pullout force divided by the sidewall interface area of the plug (Eq. 23). Results of pullout tests were included in the database of McVay et al. (1992) that forms the basis of Eq. 33, and McVay et al. recommend the test as an alternative method for estimating side resistance for design.

Cohesionless IGM The FHWA *Drilled Shaft Manual* (O'Neill and Reese 1999) recommends a procedure for calculating unit side resistance specifically for cohesionless IGM. These are granular materials exhibiting SPT N_{60}-values between 50 and 100. The method follows the general approach for calculating side resistance of drilled shafts in granular soils, given by

$$f_{su} = \sigma'_v K_o \tan\phi' \tag{35}$$

in which f_{su} = ultimate unit side resistance, σ_v' = vertical effective stress, K_o = in situ coefficient of lateral earth pressure, and ϕ' = effective stress friction angle of the IGM. The modifications to account for cohesionless IGM behavior are incorporated into empirical correlations with the N-value as follows:

$$\sigma'_p = 0.2 N_{60} p_a \tag{36}$$

$$OCR = \frac{\sigma'_p}{\sigma'_v} \tag{37}$$

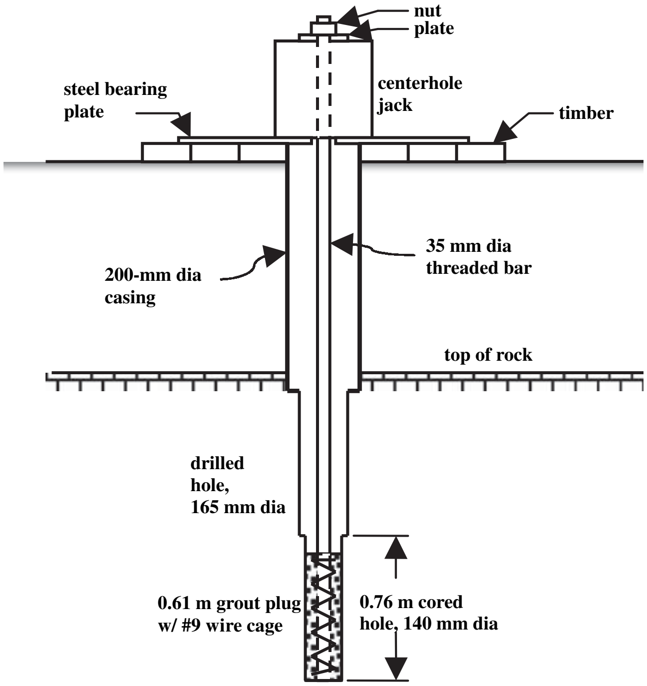


FIGURE 26 Small-scale pullout test used in Florida limestone (after Crapps 1986).

$$\phi' = \tan^{-1}\left[\frac{N_{60}}{12.2 + 20.3\left(\dfrac{\sigma'_v}{p_a}\right)}\right]^{0.34} \tag{38}$$

$$K_o = (1 - \sin\phi')\mathrm{OCR}^{\sin\phi'} \tag{39}$$

where σ'_p = preconsolidation stress, σ'_v = average vertical effective stress over the layer, N_{60} = SPT N-value corresponding to 60% hammer efficiency, and OCR = overconsolidation ratio. O'Neill et al. (1996) reported good agreement with results of load tests on shafts in residual micaceous sands in the Piedmont province (Harris and Mayne 1994) and granular glacial till in the northeastern United States.

Soft Argillaceous Rock Side resistance in weak shales and claystones can be approximated for design using the relationships given previously for rock. Alternatively, a procedure for evaluating unit side resistance specifically in argillaceous (containing clay) cohesive IGMs is presented in the FHWA *Drilled Shaft Manual* (O'Neill and Reese 1999). The ultimate unit side resistance is given by

$$f_{su} = \alpha q_u \tag{40}$$

where q_u = compressive strength of intact rock and α = empirical factor given in Figure 27. In Figure 27, σ_n = fluid pressure exerted by the concrete at the time of the pour and σ_p = atmospheric pressure in the same units as σ_n. As indicated in Figure 27, the method is based on an assumed value of interface friction angle ϕ_{rc} = 30 degrees. If it is known that a

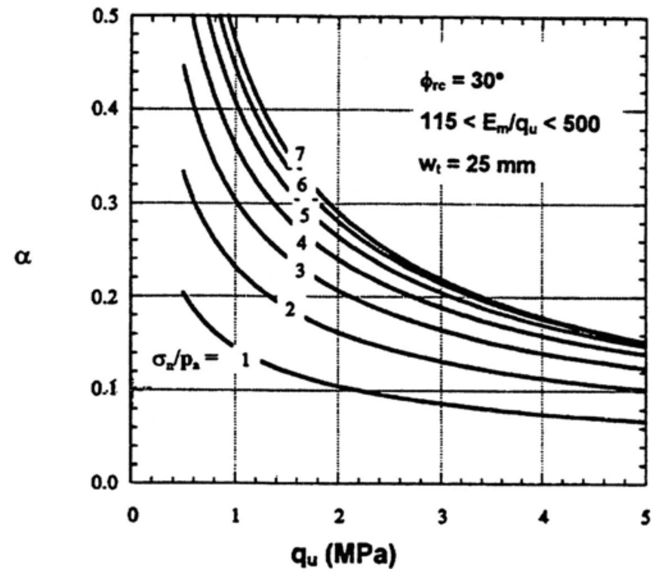


FIGURE 27 Factor α for cohesive IGM (O'Neill et al. 1996).

different value of interface friction applies, then the parameter α can be adjusted by

$$\alpha = \alpha_{26}\frac{\tan\phi_{rc}}{\tan 30^\circ} \tag{41}$$

The method is based on work reported by Hassan et al. (1997) in which detailed modeling and field testing were conducted to study side resistance of shafts in clay–shales of Texas. O'Neill and Reese (1999) provide additional equations for modifying side resistance for roughness, the presence of soft seams, and other factors, and the reader is advised to consult their work for these additional details.

Correlations with In Situ Tests

The survey responses indicate that several states use measurements from field penetration tests to estimate unit side resistance in weak rock. As an example, consider the Texas cone penetration test (TCPT) described in chapter two. The Texas DOT *Geotechnical Manual*, which is accessible online (2005), presents graphs for estimating design values of allowable unit side and base resistances as a function of the penetration resistance (millimeters of penetration per 100 blows), for materials that exhibit TCPT blowcounts of greater than 100 blows per 300 mm. For materials exhibiting fewer than 100 blowcounts, separate graphs are provided for allowable values of unit side and base resistances. The Missouri DOT also reports using the TCPT to correlate allowable side and base resistances in weak rock.

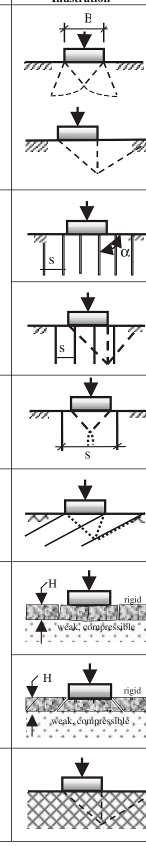
Base Resistance

Load transmitted to the base of a rock-socketed shaft, expressed as a percentage of the axial compression load applied at the head, can vary over a wide range at typical working loads. Several authors suggest a typical range of 10% to 20% of the head load (Williams et al. 1980; Carter and Kulhawy 1988), and some authors suggest that base resistance should be neglected entirely for rock-socket design (Amir 1986). Elasticity solutions show that base load transfer depends on the embedment ratio (L/B) and the modulus ratio (E_c/E_r). The ratio of base load to applied load (Q_b/Q_c) decreases with increasing L/B (see Figure 21) and increases with increasing modular ratio. As discussed previously, there is ample evidence that base resistance should not be discounted in most cases (Figure 23), and that construction and inspection methods are available to control base quality. Load tests, described in chapter five, provide a means to determine the effects of construction on base load transfer.

The ultimate base resistance of a rock-socketed drilled shaft, Q_b, is the product of the limiting normal stress, or bearing capacity, q_{ult}, at the base and the cross-sectional area of the shaft base (A_b):

$$Q_b = q_{ult}\, A_b = q_{ult}\, [1/4\ \pi B^2] \tag{42}$$

Analytical solutions for bearing capacity of rock are based on the general bearing capacity equation developed for soil, with appropriate modifications to account for rock mass characteristics such as spacing and orientation of discontinuities, condition of the discontinuities, and strength of the rock mass. Typical failure modes for foundations bearing on rock are shown in Figure 28. The failure modes depicted were intended to address shallow foundations bearing on rock (Sowers 1976); however, the general concepts should be applicable to bearing capacity of deep foundations. The cases shown can be placed into four categories: massive, jointed, layered, and fractured rock.



Rock Mass Conditions			Failure		
	Joint Dip Angle, from horizontal	Joint Spacing	Illustration	Mode	Bearing Capacity Equation No.
INTACT / MASSIVE	N/A	S>>B		(a) Brittle Rock: Local shear failure caused by localized brittle fracture	Eq. 43
				(b) Ductile Rock: General shear failure along well-defined shear surfaces	Eq. 43
STEEPLY DIPPING JOINTS	$70^\circ < \alpha < 90^\circ$	S < B		(c) Open Joints: Compression failure of individual rock columns	Eq. 44
				(d) Closed Joints General shear failure along well defined failure surfaces; near vertical joints	Eqs. 45–52
		S > B		(e) Open or Closed Joints: Failure initiated by splitting leading to general shear failure; near vertical joints	Eqs. 53–54
JOINTED	$20^\circ < \alpha < 70^\circ$	S < B or S > B if failure we dge can de ve lop along joints		(f) General shear failure with potential for failure along joints; moderately dipping joint sets.	Eqs. 45–52
LAYERED	$0 < \alpha < 20^\circ$	Limiting va lue of H w/re to B is dependent upon material properties		(g) Rigid layer over weak compressible layer: Failure is initiated by tensile failure caused by flexure of rigid upper layer	N/A
				(h) Thin rigid layer over weak compressible layer: Failure is by punching shear through upper layer	N/A
FRACTURED	N/A	S << B		(g) General shear failure with irregular failure surface through fractured rock mass; two or more closely spaced joint sets	Eq. 57

FIGURE 28 Bearing capacity failure modes in rock (after *Rock Foundations* 1994).

Massive Rock

For this case, the ultimate bearing capacity will be limited to the bearing stress that causes fracturing in the rock. An intact rock mass can be defined, for purposes of bearing capacity analysis, as one for which the effects of discontinuities are insignificant. Practically, if joint spacing is more than four to five times the shaft diameter, the rock is massive. If the base is embedded in rock to a depth of at least one diameter, the failure mode is expected to be by punching shear (Figure 28, mode a). In this case, Rowe and Armitage (1987b) stated that rock fracturing can be expected to occur when the bearing stress is approximately 2.7 times the rock uniaxial compressive strength. For design, the following is recommended:

$$q_{ult} = 2.5\ q_u \tag{43}$$

Other conditions that must be verified are that the rock to a depth of at least one diameter below the base of the socket is either intact or tightly jointed (no compressible or gouge-filled seams) and there are no solution cavities or voids below the base of the pier. O'Neill and Reese (1999) recommend limiting base resistance to $2q_u$ if the embedment into rock is less than one diameter. In rock with high compressive strength, the designer also must determine the structural capacity of the shaft, which may govern the allowable normal stress at the base.

Jointed Rock Mass

When discontinuities are vertical or nearly vertical ($\alpha > 70^\circ$), and open joints are present with a spacing less than the socket diameter ($S < B$, Figure 28, mode c), failure can occur (theoretically) by unconfined compression of the poorly constrained columns (Sowers 1979). Bearing capacity can be estimated from

$$q_{ult} = q_u = 2c \tan\,(45^\circ + \tfrac{1}{2}\,\phi) \tag{44}$$

where q_u = uniaxial compressive strength and c and ϕ are Mohr–Coulomb strength properties of the rock mass. If the nearly vertical joints are closed (Figure 28, mode d), a general wedge failure mode may develop and the bearing capacity can be approximated using Bell's solution for plane strain conditions:

$$q_{ult} = cN_c s_c + \frac{B}{2}\gamma N_\gamma s_\gamma + \gamma D N_q s_q \qquad (45)$$

in which B = socket diameter; γ = effective unit weight of the rock mass; D = foundation depth; N_c, N_γ, and N_q are bearing capacity factors; and s_c, s_γ, and s_q are shape factors to account for the circular cross section. The bearing capacity factors and shape factors are given by:

$$N_c = 2\sqrt{N_\phi}\,\left(N_\phi + 1\right) \qquad (46)$$

$$N_\gamma = \sqrt{N_\phi}\,\left(N_\phi^2 - 1\right) \qquad (47)$$

$$N_q = N_\phi^2 \qquad (48)$$

$$N_\phi = \tan^2\left(45^o + \frac{\phi}{2}\right) \qquad (49)$$

$$s_c = 1 + \frac{N_q}{N_c} \qquad (50)$$

$$s_\gamma = 0.6 \qquad (51)$$

$$s_q = 1 + \tan\phi \qquad (52)$$

In these equations (46–52), the values of c and ϕ are RMS properties, which may be difficult to determine accurately for rock mass beneath the base of drilled shafts.

If joint spacing S is greater than the socket diameter (Figure 28, mode e), failure occurs by splitting, leading eventually to general shear failure. This problem has been evaluated by Bishnoi (1968) and developed further by Kulhawy and Goodman (1980). The solution can be expressed by:

$$q_{ult} = J\ c\ N_{cr} \qquad (53)$$

in which J = a correction factor that depends on the ratio of horizontal discontinuity spacing to socket diameter (H/B) as shown in Figure 29, c = rock mass cohesion, and N_{cr} = a bearing capacity factor given by

$$N_{cr} = \frac{2N_\phi^2}{1+N_\phi}\left(\cot\phi\right)\frac{S}{B}\left(1 - \frac{1}{N_\phi}\right) - N_\phi\left(\cot\phi\right) + 2\sqrt{N_\phi} \qquad (54)$$

where N_ϕ is given by Eq. 49. If the actual RMS properties are not evaluated, Kulhawy and Carter (1992a) suggest that rock mass cohesion in Eq. 53 can be approximated as $0.1q_u$, where q_u = uniaxial compressive strength of intact rock. Rock mass cohesion can also be estimated from the Hoek–Brown strength properties using Eq. 17 given in chapter two.

For the case of moderately dipping joint sets (Figure 28, mode f, $20^\circ < \alpha < 70^\circ$), the failure surface is likely to develop along the discontinuity planes. Eq. 45 can be used, but with strength parameters representative of the joints. *Rock Foundations* (1994) recommends neglecting the first term in Eq. 45 based on the assumption that the cohesion component of strength along the joint surfaces is highly uncertain.

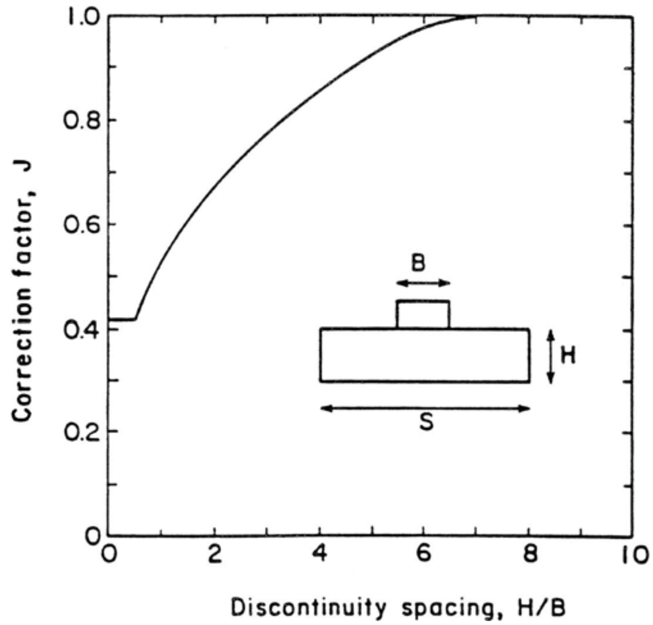


FIGURE 29 Correction factor for discontinuity spacing (Kulhawy and Carter 1992a).

Layered Rocks

Sedimentary rock formations often consist of alternating hard and soft layers. For example, soft layers of shale interbedded with hard layers of sandstone. Assuming the base of the shaft is bearing on the more rigid layer, which is underlain by a soft layer, failure can occur either by flexure if the rigid layer is relatively thick or by punching shear if the rigid layer is thin (Figure 28, modes g and h). Both modes are controlled fundamentally by the tensile strength of the intact rock, which can be approximated as being on the order of 5% to 10% of the uniaxial compressive strength. According to Sowers (1979) neither case has been studied adequately and no analytical solution proposed. The failure modes depicted in Figure 28 merely suggest possible methods for analysis.

Fractured Rock Mass

A rational approach for calculating ultimate bearing capacity of rock masses that include significant discontinuities (Figure 28, mode g) is to apply Eq. 45 with appropriate RMS properties, c' and ϕ'. However, determination of c' and ϕ' for highly fractured rock mass is not straightforward because the failure envelope is nonlinear and there is no standard test method for direct measurement. One possible approach is to employ the Hoek–Brown strength criterion described in chapter two. The criterion is attractive because (1) it captures the nonlinearity in the strength envelope that is observed in jointed rock masses and (2) the required parameters can be estimated empirically using correlations to GSI and RMR, also described in chapter two. To use this approach, it is necessary to relate the Hoek–Brown strength parameters (m_b, s, and a) to Mohr–Coulomb strength parameters (c' and ϕ'); for example, using Eqs. 16 and 17 in chapter two.

Alternatively, several authors (Carter and Kulhawy 1988; Wyllie 1999) have shown that a conservative, lower-bound estimate of bearing capacity can be made directly in terms of Hoek–Brown strength parameters by assuming a failure mode approximated by active and passive wedges; that is, the Bell solution for plane strain. The failure mass beneath the foundation is idealized as consisting of two zones, as shown in Figure 30. The active zone (Zone 1) is subjected to a major principal stress (σ'_1) coinciding at failure with the ultimate bearing capacity (q_{ult}) and a minor principal stress (σ'_3) that satisfies equilibrium with the horizontal stress in the adjacent passive failure zone (Zone 2). In Zone 2, the minor principal stress is vertical and conservatively assumed to be zero, whereas the major principal stress, acting in the horizontal direction, is the ultimate strength according to the Hoek–Brown criterion. From chapter two, the strength criterion is given by

$$\sigma'_1 = \sigma'_3 + q_u \left(m_b \frac{\sigma'_3}{q_u} + s \right)^a \tag{55}$$

where σ'_1 and σ'_3 = major and minor principal effective stresses, respectively; q_u = uniaxial compressive strength of intact rock; and m_b, s, and a are empirically determined strength parameters for the rock mass. For Zone 2, setting the vertical stress $\sigma'_3 = 0$ and solving Eq. 55 for σ'_1 yields

$$\sigma'_1 = \sigma'_H = q_u s^a \tag{56}$$

where σ'_H = horizontal stress in Zone 2. To satisfy equilibrium, the horizontal stress given by Eq. 56 is set equal to σ'_3 in Zone 1. Substituting $\sigma'_3 = q_u s^{0.5}$ into Eq. 55 and considering that $\sigma'_1 = q_{ult}$ yields

$$q_{ult} = q_u \left[s^a + \left(m_b s^a + s \right)^a \right] \tag{57}$$

The assumption of zero vertical stress at the bearing elevation may be overly conservative for many rock sockets. A similar derivation can be carried out with the overburden stress taken into account, resulting in the following. Let

$$A = \sigma'_{v,b} + q_u \left[m_b \frac{\left(\sigma'_{v,b}\right)}{q_u} + s \right]^a \tag{58}$$

where $\sigma'_{v,b}$ = vertical effective stress at the socket bearing elevation, which is also the minor principal stress in Zone 2. Then

$$q_{ult} = A + q_u \left[m_b \left(\frac{A}{q_u} \right) + s \right]^a \tag{59}$$

A limitation of Eqs. 57–59 is that they are based on the assumption of plane strain conditions, corresponding to a strip footing. Kulhawy and Carter (1992a) noted that for a circular foundation the horizontal stress between the two assumed failure zones may be greater than for the plane strain case, resulting in higher bearing capacity. The analysis is therefore conservative for the case of drilled shafts.

Eqs. 57–59 require determination of a single rock strength property (q_u) along with an approximation of the Hoek–Brown strength parameters. In chapter two, the Hoek–Brown strength parameters are correlated to GSI by Eqs. 12–15. This allows a correlation to be made between the GSI of a rock mass; the value of the coefficient m_i for intact rock as given in Table 11 (chapter two), and the bearing capacity ratio q_{ult}/q_u by Eq. 57. The resulting relationship is shown graphically in Figure 31. The bearing capacity ratio is limited by an upper-bound value of 2.5, corresponding to the recommendation of Rowe and Armitage (Eq. 43).

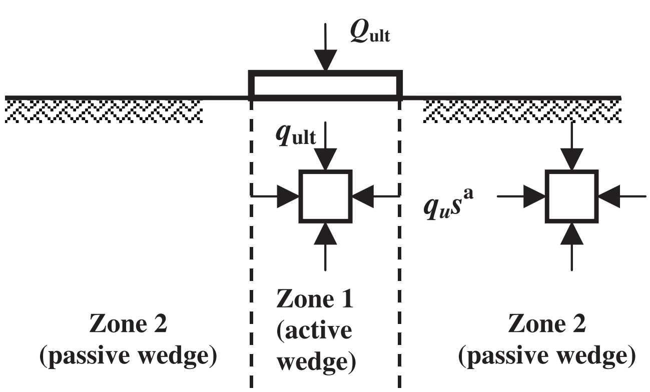


FIGURE 30 Bearing capacity analysis.

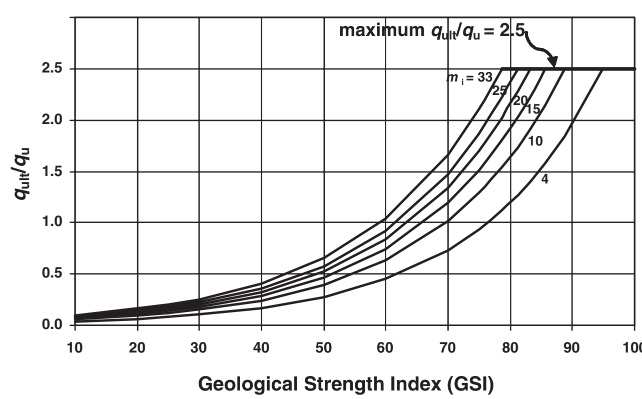


FIGURE 31 Bearing capacity ratio versus GSI.

Alternatively, the bearing capacity ratio can be related approximately to the rock mass description based on RMR (Table 9) using an earlier correlation given by Hoek and Brown (1988). The resulting Hoek–Brown strength parameters (m, a, and s) are substituted into Eq. 57 to obtain the bearing capacity ratio as a function of RMR. This relationship is shown graphically in Figure 32. Both figures are for the case of zero overburden stress at the bearing elevation. To account for the depth of embedment and resulting surcharge stress, Eqs. 58 and 59 can be used.

Method Based on Field Load Tests

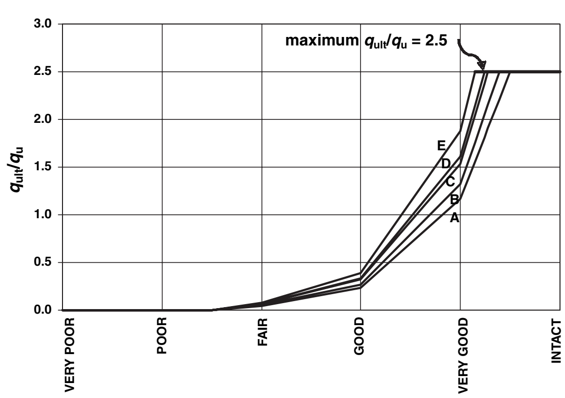
Zhang and Einstein (1998) compiled and analyzed a database of 39 load tests to derive an empirical relationship between ultimate unit base resistance (q_{ult}) and uniaxial compressive strength of intact rock (q_u). Reported values of uniaxial compressive strength ranged from 0.52 MPa to 55 MPa, although most were in the range of relatively low strength. The authors relied on the interpretation methods of the original references to determine ultimate base capacity and acknowledge that some uncertainties and variabilities are likely to be incorporated into the database as a result. The results are shown on a log–log plot in Figure 33. The linear relationship recommended by Rowe and Armitage (1987a) is shown for comparison. Based on statistical analysis of the data, the following recommendations are given by the authors:

Lower bound: $q_{ult} = 3.0\sqrt{q_u}$ (60)

Upper bound: $q_{ult} = 6.6\sqrt{q_u}$ (61)

Mean: $q_{ult} = 4.8\sqrt{q_u}$ (62)

Eqs. 60–62 provide a reasonably good fit to the available data and can be used for estimating ultimate base resistance, with due consideration of the limitations associated with predicting a rock mass behavior on the basis of a single strength parameter for intact rock. Rock mass discontinuities are not accounted for explicitly, yet they clearly must affect bearing capacity. By taking this empirical approach, however, rock mass behavior is accounted for implicitly because the load tests on which the method is based were affected by the characteristics of the rock masses. Additional limitations to the approach given by Zhang and Einstein are noted in a discussion of their paper by Kulhawy and Prakoso (1999).



A. Carbonate rocks with well-developed cleavage: *dolomite, limestone, marble*.
B. Lithified argillaceous rocks: *mudstone, siltstone, shale, and slate*.
C. Arenaceous rocks with strong crystals and poorly developed crystal cleavage: *sandstone and quartzite*.
D. Fine-grained polyminerallic igneous crystalline rocks: *andesite, dolerite, diabase, and rhyolite*.
E. Coarse-grained polyminerallic igneous and metamorphic crystalline rocks: *amphibolite, gabbro, gneiss, quartz, norite, quartz-diorite*.

FIGURE 32 Bearing capacity ratio as a function of rock type and RMR classification (see Table 9).

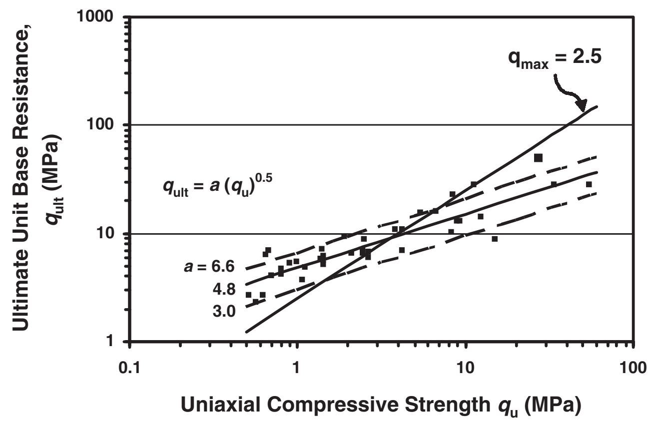


FIGURE 33 Unit base resistance versus intact rock strength (derived from Zhang and Einstein 1998).

None of the analytical bearing capacity models described above by Eqs. 44 through 59 and depicted in Figure 28 have been evaluated and verified against results of full-scale field load tests on rock-socketed drilled shafts. The primary reason for this is a lack of load test data accompanied by sufficient information on rock mass properties needed to apply the models.

Canadian Geotechnical Society Method

The *Canadian Foundation Engineering Manual* [Canadian Geotechnical Society (CGS) 1985] presents a method to estimate ultimate unit base resistance of piles or shafts bearing on rock. The CGS method is described as being applicable to sedimentary rocks with primarily horizontal discontinuities, where discontinuity spacing is at least 0.3 m (1 ft) and discontinuity aperture does not exceed 6 mm (0.25 in.). The method is given by the following:

$$q_{ult} = 3 q_u K_{sp} d \tag{63}$$

in which

$$K_{sp} = \frac{3 + \dfrac{s_v}{B}}{10\sqrt{1 + 300\dfrac{t_d}{s_v}}} \tag{64}$$

$$d = 1 + 0.4\frac{L_s}{B} \tag{65}$$

where

s_v = vertical spacing between discontinuities,
t_d = aperture (thickness) of discontinuities,
B = socket diameter, and
L_s = depth of socket (rock) embedment.

A method to calculate ultimate unit base resistance from PMT is also given by CGS as follows:

$$q_{ult} = K_b\left(p_1 - p_o\right) + \sigma_v \tag{66}$$

where

p_1 = limit pressure determined from PMT tests averaged over a depth of two diameters above and below socket base elevation,
p_o = at-rest total horizontal stress measured at base elevation,
σ_v = total vertical stress at base elevation; and
K_b = socket depth factor given as follows:

H/D	0	1	2	3	5	7
K_b	0.8	2.8	3.6	4.2	4.9	5.2

The two CGS methods described earlier are adopted in the draft 20006 Interim AASHTO *LRFD Bridge Design Specifications* (2006).

AXIAL LOAD-DISPLACEMENT BEHAVIOR

Analysis of the load-displacement behavior of a drilled shaft is an essential step in a rational design. Design of most sockets is governed by the requirement to limit settlement to a specified allowable value. The problem of predicting vertical displacement at the top of a rock socket has been studied through theoretical and numerical analyses along with limited results from full-scale field load testing. Methods that appear to have the most application to design of highway bridge foundations are summarized in this section.

The basic problem is depicted in Figure 34 and involves predicting the relationship between an axial compression load (Q_c) applied to the top of a socketed shaft and the resulting axial displacement at the top of the socket (w_c). The concrete shaft is modeled as an elastic cylindrical inclusion embedded within an elastic rock mass. The cylinder of depth L and diameter B has Young's modulus E_c and Poisson's ratio v_c. The rock mass surrounding the cylinder is homogeneous with Young's modulus E_r and Poisson's ratio v_c, whereas the rock mass beneath the base of the shaft has Young's modulus E_b and Poisson's ratio v_b. (Note: some authors use E_r to denote modulus of rock in elasticity solutions; elsewhere in this report, E_r denotes modulus of intact rock and E_M is the rock mass modulus of deformation.) The

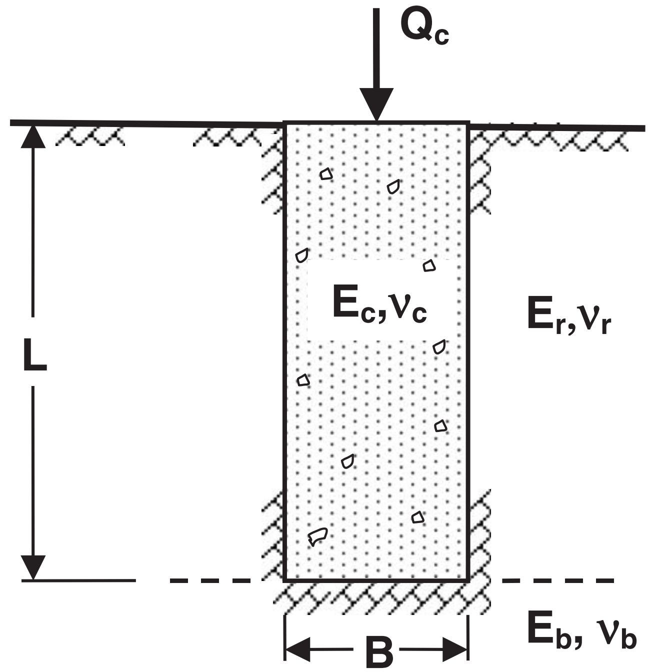


FIGURE 34 Axially loaded rock socket, elastic analysis.

shaft is subjected to a vertical compressive force Q_c assumed to be uniformly distributed over the cross-sectional area of the shaft resulting in an average axial stress $\sigma_b = 4Q/(\pi B^2)$.

Early solutions to the problem of a single compressible pile in an elastic continuum were used primarily to study the response of deep foundations in soil (e.g., Mattes and Poulos 1969; Butterfield and Banerjee 1971; Randolph and Wroth 1978). In most cases, the solutions were not directly applicable to rock sockets because they did not cover the typical ranges of modulus ratio (E_c/E_r) or embedment ratio (L/B) of rock sockets, but they did provide the basic methodology for analysis of the problem. Osterberg and Gill (1973) used an elastic finite-element formulation to analyze rock sockets with D/B ranging from zero to 4 and the modulus ratios ranging from 0.25 to 4. Their analysis also considered differences between the modulus of the rock beneath the base (E_b) and that along the shaft (E_r). Results showed the influence of these parameters on load transfer, in particular the relative portion of load carried in side resistance and transmitted to the base, but did not provide a method for predicting load-displacement behavior for design. Pells and Turner (1979) and Donald et al. (1980) conducted finite-element analyses assuming elastic and elastoplastic behaviors. Their numerical results were used to determine values of the dimensionless influence factor (I_ρ) that can be used to predict elastic deformation using the general equation

$$w_c = \frac{Q_c}{E_r B} I_\rho \tag{67}$$

Values of the influence factor were presented in the form of charts for a range of modulus and embedment ratios common for rock sockets. Graphs are also provided showing the ratio of applied load transferred to the base (Q_b/Q_c). These studies provided the first practical methods for predicting the load-displacement response of rock sockets. Their principal limitation lies in the assumption of a full bond between the shaft and the rock; that is, no slip. Observations from load tests; for example, Horvath et al. (1983), show that peak side resistance may be reached at displacements on the order of 5 mm. Rupture of the interface bond begins at this point, resulting in relative displacement (slip) between the shaft and surrounding rock. Under service load conditions, most rock sockets will undergo displacements that reach or exceed the full slip condition and should be designed accordingly. Analyses that account for both fully bonded conditions and full slip conditions provide a more realistic model of load-displacement response.

Rowe and Pells (1980) conducted a theoretical study based on finite-element analyses of rock-socketed shafts that accounts for the possibility of slip at the shaft–rock interface. The analysis treats the shaft and rock as elastic materials, but provides for plastic failure within the rock or the concrete shaft and for slip at the cohesive-frictional and dilatant rock–shaft interface. At small loads, the shaft, rock, and interface are linearly elastic and the shaft is fully bonded to the rock. Slip is assumed to occur when the mobilized shear stress reaches the interface strength, assumed to be governed by a Mohr–Coulomb failure criterion:

$$\tau = c_{\text{peak}} + \sigma_n \tan\phi_{\text{peak}} \tag{68}$$

where c_{peak} = peak interface adhesion, σ_n = interface normal stress, and ϕ_{peak} = peak interface friction angle. Once slip occurs, it is assumed that c and ϕ degrade linearly with relative displacement between the two sides of the ruptured interface from the peak values to residual values (c_{residual}, ϕ_{residual}) at a relative displacement δ_r. Roughness of the interface is modeled in terms of a dilatancy angle ψ and a maximum dilation, and strain softening of the interface is considered. Modeling of the interface in this way provides a good mechanistic representation of the load-displacement behavior of a rock-socketed shaft, as described in the beginning of this chapter (Figure 19).

From these studies, Rowe and Armitage (1987a,b) prepared design charts that enable construction of a theoretical load-displacement curve in terms of (1) the influence factor I_ρ used to calculate axial displacement by Eq. 67 and (2) the ratio of load Q_b/Q_t transmitted to the socket base, where Q_b = base load and Q_t = total load applied to the top of the shaft. Figure 35 is an example of the chart solution for a complete socket. The charts offer a straightforward means of calculating load-displacement curves and have been used by practitioners for the design of bridge foundations. The Rowe and Armitage solutions represent a standard against which approximate methods can be compared and verified.

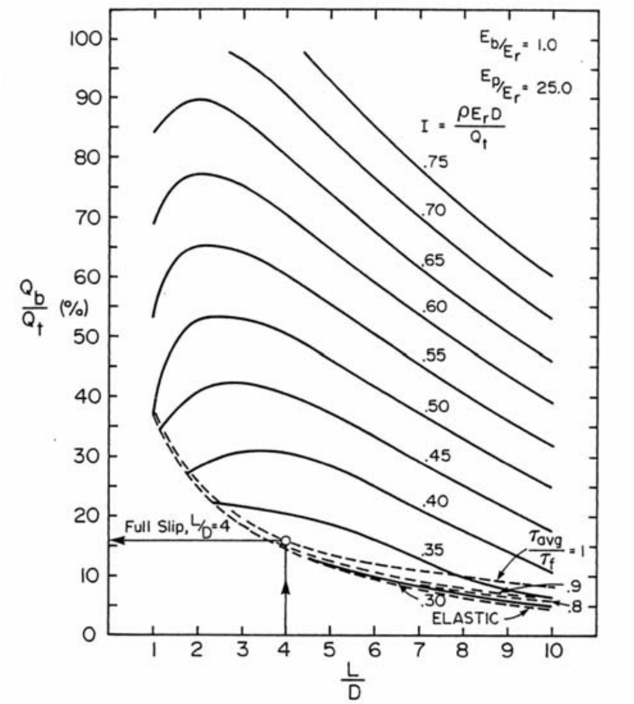


FIGURE 35 Design chart for shaft displacement and base load transfer, complete socket (Rowe and Armitage 1987a).

Rowe and Armitage (1987a,b) developed design charts for two contact conditions at the base of the socket: (1) a "complete socket," for which full contact is assumed between the base of the concrete shaft and the underlying rock; and (2) a shear socket, for which a void is assumed to exist beneath the base. These conditions are intended to model the socket arrangements and methods of loading used in field load testing. When clean base conditions during construction can be verified and instrumentation is provided for measuring base load, a complete socket is assumed. Frequently, however, base resistance is eliminated by casting the socket above the base of the drilled socket, in which case the test shaft is modeled as a shear socket. The boundary condition at the base of a shear socket under axial compression is one of zero stress. The charts given in Rowe and Armitage (1987a,b) provide a rigorous method for analyzing rock-socket load-displacement behavior.

Closed Form Solutions

An approximate method given by Kulhawy and Carter (1992b) provides simple, closed-form expressions that are attractive for design purposes and yield results that compare well with those of Rowe and Armitage. For axial compression loading, the two cases of complete socket and shear socket are treated. Solutions were derived for two portions of the load-displacement curve depicted in Figure 19; the initial linear elastic response (OA) and the full slip condition (region beyond point B). The closed-form expressions cannot predict the load-displacement response between the occurrence of first slip and full slip of the shaft (AB). However, the nonlinear finite-element results indicate that the progression of slip along the socket takes place over a relatively small interval of displacement. Comparisons of the bilinear curve given by the closed-form expressions with results of Rowe and Armitage (1987b) indicated that this simplification is reasonably accurate for the range of rock-socket conditions encountered in practice.

The closed-form expressions for approximating the load-displacement curves for complete and shear socket are given here. For a full description of the assumptions and derivations the reader is referred to Carter and Kulhawy (1988) and Kulhawy and Carter (1992b).

1. For the linearly elastic portion of the load-displacement curve.
   (a) Shear socket (zero stress at the base):

$$\frac{E_r B w_c}{2Q_c} = \left(\frac{1}{\pi}\right)\left(\frac{2}{\mu B}\right)\left(\frac{E_r}{E_c}\right)\left(\frac{\cosh[\mu L]}{\sinh[\mu L]}\right) \quad (69)$$

in which w_c = downward vertical displacement at the butt (top) of the shaft and where μ is defined by:

$$(\mu L)^2 = \left(\frac{2}{\zeta\lambda}\right)\left(\frac{2L}{B}\right)^2 \quad (70)$$

$$\zeta = \ln\,[5(1 - v_r)L/B] \quad (71)$$

$$\lambda = E_c/G_r \quad (72)$$

$$G_r = E_r/\,[2(1 + v_r] \quad (73)$$

where G_r = elastic shear modulus of rock mass.

   (b) Complete socket:

$$\frac{G_r B w_c}{2Q_c} = \frac{1+\left(\frac{4}{1-\nu_b}\right)\left(\frac{1}{\pi\lambda\xi}\right)\left(\frac{2L}{B}\right)\left(\frac{\tanh[\mu L]}{\mu L}\right)}{\left(\frac{4}{1-\nu_b}\right)\left(\frac{1}{\xi}\right)+\left(\frac{2\pi}{\zeta}\right)\left(\frac{2L}{B}\right)\left(\frac{\tanh[\mu L]}{\mu L}\right)} \quad (74)$$

where

$$\xi = G_r/G_b \quad (75)$$

$$G_b = E_b/\,[2(1 + v_b] \quad (76)$$

The magnitude of load transferred to the base of the shaft (Q_b) is given by

$$\frac{Q_b}{Q_c} = \frac{\left(\frac{4}{1-\nu_b}\right)\left(\frac{1}{\xi}\right)\left(\frac{1}{\cosh[\mu L]}\right)}{\left(\frac{4}{1-\nu_b}\right)\left(\frac{1}{\xi}\right)+\left(\frac{2\pi}{\zeta}\right)\left(\frac{2L}{B}\right)\left(\frac{\tanh[\mu L]}{\mu L}\right)} \quad (77)$$

2. For the full slip portion of the load-displacement curve.
   (a) Shear socket:

$$w_c = F_1\left(\frac{Q_c}{\pi E_r B}\right) - F_2 B \quad (78)$$

in which $F_1 = a_1(\lambda_2 BC_2 - \lambda_1 BC_1) - 4a_3$ (79)

$$F_2 = a_2 \left(\frac{c}{E_r}\right) \quad (80)$$

$$C_{1,2} = \exp[\lambda_{2,1}L]/(\exp[\lambda_2 L] - \exp[\lambda_1 L]) \quad (81)$$

$$\lambda_{1,2} = \frac{-\beta \pm (\beta^2 + 4\alpha)^{1/2}}{2\alpha} \quad (82)$$

$$\alpha = a_1\left(\frac{E_c}{E_r}\right)\left(\frac{B^2}{4}\right) \quad (83)$$

$$\beta = a_3\left(\frac{E_c}{E_r}\right)B \quad (84)$$

$$a_1 = (1 + v_r)\zeta + a_2 \quad (85)$$

$$a_2 = \left[(1-\nu_c)\left(\frac{E_r}{E_c}\right)+(1+\nu_r)\right]\left(\frac{1}{2\tan\phi\tan\psi}\right) \quad (86)$$

$$a_3 = \left(\frac{\nu_c}{2\tan\psi}\right)\left(\frac{E_r}{E_c}\right) \quad (87)$$

   (b) Complete socket:

$$w_c = F_3\left(\frac{Q_c}{\pi E_r B}\right) - F_4 B \quad (88)$$

in which

$$F_3 = a_1(\lambda_1 BC_3 - \lambda_2 BC_4) - 4a_3 \quad (89)$$

$$F_4 = \left[1 - a_1 \left(\frac{\lambda_1 - \lambda_2}{D_4 - D_3}\right) B\right] a_2 \left(\frac{c}{E_r}\right) \quad (90)$$

$$C_{3,4} = \frac{D_{3,4}}{D_4 - D_3} \quad (91)$$

$$D_{3,4} = \left[\pi(1-\nu_b^2)\left(\frac{E_r}{E_b}\right) + 4a_3 + a_1\lambda_{2,1}B\right]\exp[\lambda_{2,1}L] \quad (92)$$

The magnitude of load transferred to the base of the shaft (Q_b) is given by

$$\frac{Q_b}{Q_c} = P_3 + P_4\left(\frac{\pi B^2 c}{Q_c}\right) \quad (93)$$

in which

$$P_3 = a_1(\lambda_1 - \lambda_2)\,B\,\exp[(\lambda_1 + \lambda_2)L]/(D_4 - D_3) \quad (94)$$

$$P_4 = a_2(\exp[\lambda_2 D] - \exp[\lambda_1 L])/(D_4 - D_3) \quad (95)$$

The solutions given previously (Eqs. 69–95) are easily implemented by spreadsheet, thus providing designers with a simple analytical tool for assessing the likely ranges of behavior for trial designs. A spreadsheet solution provides the opportunity to easily evaluate the effects of various input parameters on load-displacement response. When combined with appropriate judgment and experience, this approach represents a reasonable analysis of rock-socketed drilled shafts. The method of Carter and Kulhawy presented herein is also adopted in the FHWA *Drilled Shaft Manual* (O'Neill and Reese 1999) for analysis of load-displacement response of single drilled shafts in rock (see Appendix C of the *Manual*). Reese and O'Neill also present methods for predicting load-displacement response of shafts in cohesive IGMs and cohesionless IGMs. The equations are not reproduced here, but are given as closed-form expressions that can be implemented easily using a spreadsheet.

Other Methods

A computer program that models the axial load-displacement behavior of a rock-socketed shaft, based on the methods described by Seidel and Haberfield (1994) and described briefly earlier, has been developed. The program ROCKET requires the following input parameters:

- Drained shear strength parameters of the intact rock,
- Rock mass modulus and Poisson's ratio,
- Foundation diameter,
- Initial normal stress, and
- Mean socket asperity length and mean socket asperity angle.

Although some of these parameters are determined on a routine basis for the design of rock-socketed foundations, asperity characteristics are not typically evaluated. Drained triaxial tests are also not considered routine by most transportation agencies in the United States. However, this approach is promising because it provides a theoretical basis for predicting rock-socket behavior that encompasses more of the important parameters than the empirical approaches now available to predict side resistance.

Combining Side and Base Resistances

A fundamental aspect of drilled shaft response to axial compression loading is that side and base resistances are mobilized at different downward displacements. Side resistance typically reaches a maximum at relatively small displacement, in the range of 5 to 10 mm. Beyond this level, side resistance may remain constant or decrease, depending on the stress–strain properties of the shaft-rock interface. Ductile behavior describes side resistance that remains constant or decreases slightly with increasing displacement. If the interface is brittle, side resistance may decrease rapidly and significantly with further downward displacement. One question facing the designer is how much side resistance is mobilized at the strength limit state. As stated by O'Neill and Reese (1999), "the issue of whether ultimate side resistance should be added directly to the ultimate base resistance to obtain an ultimate value of resistance is a matter of engineering judgment." Responses to Question 20 of the survey (Appendix A) show a wide range in the way that side and base resistances are combined for design of rock sockets. Several states indicated that they follow the guidelines given by O'Neill and Reese in the FHWA *Drilled Shaft Manual* (1999). Three possible approaches are described here. The first applies to the case where load testing or laboratory shear strength tests prove that the rock is ductile. In this case, the ultimate values of side and base resistance are added directly. If no field or lab tests are conducted, a "fully reduced frictional shearing resistance at the interface" is used to compute side resistance and this value is added to the ultimate base resistance. The fully reduced strength is taken as the residual shear strength of the rock = $\sigma'_h \tan\phi_{rc}$, where σ'_h = horizontal effective stress normal to the interface and ϕ_{rc} = residual angle of interface friction between rock and concrete. A value of 25 degrees is recommended in the absence of measurements and σ'_h is assumed to be equal to the vertical effective stress σ'_v. A second approach is recommended for cases in which base resistance is neglected in design. In this case, the ultimate side resistance is recommended for design at the strength limit state, unless "progressive side shear failure could occur," in which case the resistance should be reduced "according to the judgment of the geotechnical engineer."

Several states indicate in their response to Question 20 that load testing, especially using the Osterberg load cell, is one of the ways in which the issue is addressed. Load tests that provide independent measurements of side and base resistance as a function of displacement and that are carried to large displacements provide the best available data for establishing resistance values. When testing is not conducted, the major uncertainty (i.e., judgment) is associated with the question of whether or not side shear behavior will exhibit significant strain softening. Research is needed to provide guidance on what conditions are likely to produce a "brittle" response of the side resistance. Most load test data in which side resistance is measured independently do not exhibit a severe decrease in side resistance with large displacement. A study with the objective of identifying the factors that control stress–strain behavior at the shaft–rock interface at large displacement is needed. A large amount of data have been produced in recent years from load tests using the Osterberg cell (O-cell) and such data would be useful for a study of post-peak interface behavior. The results would be most useful to practitioners if guidance could be provided on specific rock types, drilling methods, construction practices, and ranges of confining stress that are likely to cause strain softening at the interface. These factors could then be used as indicators of cases for which further field or laboratory testing is warranted.

A promising technique for improving base load-displacement response of drilled shafts involves post-grouting at the base (base grouting). The technique involves casting drilled shafts with a grout delivery system incorporated into the reinforcing cage capable of placing high pressure grout at the base of the shaft, after the shaft concrete has cured. The effect is to compress debris left by the drilling process, thus facilitating mobilization of base resistance within service or displacement limits. According to Mullins et al. (2006), base grouting is used widely internationally, but its use in North America has been limited. An additional potential advantage is that the grouting procedure allows a proof test to be conducted on the shaft. Base grouting warrants further consideration as both a quality construction technique and a testing tool for rock-socketed shafts.

For evaluation of service limit states, both side and base resistances should be included in the analysis. Analytical methods that can provide reasonable predictions of axial load-deformation response, for example the Carter and Kulhawy method described in this chapter or similar methods given by O'Neill and Reese (1999), provide practical tools for this type of analysis. All of these methods require evaluation of rock mass modulus.

CURRENT AASHTO PRACTICE

The draft 2006 Interim AASHTO *LRFD Bridge Design Specifications* recommends specific methods and associated resistance factors for evaluating side and base resistance of rock-socketed shafts under axial load. These are summarized in Table 20. The resistance factors are based on a calibration study conducted by Paikowsky et al. (2004a) and additional recommendations given by Allen (2005). Rock mass properties used with LRFD resistance factors should be based on average values, not minimum values.

Three methods are cited for predicting ultimate unit side resistance in rock. The first is identified as Horvath and Kenney (1979). However, the equation given in the AASHTO *Specifications* is actually the original Horvath and Kenney recommendation (Eq. 25), but with unit side resistance modified to account for RQD. A reduction factor, α, is applied, as determined by Table 13 and Table 18 of this report. This approach was recommended by O'Neill and Reese (1999). The second method is identified as Carter and Kulhawy (1988). The draft 2006 Interim AASHTO *LRFD Bridge Design Specifications* does not state explicitly the equation to be used in connection with the Carter and Kulhawy method. However, in the calibration study by Paikowsky et al. (2004a) the expression used for all evaluations attributed to Carter and Kulhawy is

$$f_{su} = 0.15\, q_u \tag{96}$$

in which q_u = uniaxial compressive strength of rock. In their original work, Carter and Kulhawy (1988) proposed the use of 0.15 q_u as a design check, whereas the AASHTO *Specifications* treat it as a design recommendation. This unintended usage is inappropriate and does not adequately represent the most up-to-date research based on regression analysis of the available data on socket-side resistance. The third method given by AASHTO is O'Neill and Reese (1999). It is not clear how this differs from the Horvath and Kenney (1979) method because the equations given by AASHTO are all taken directly from O'Neill and Reese (1999). The equations

TABLE 20
SUMMARY OF CURRENT AASHTO METHODS AND RESISTANCE FACTORS

Method/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts	Side resistance in rock	1. Horvath and Kenney (1979)	0.55
		2. Carter and Kulhawy (1988)	0.50
		3. ONeill and Reese (1999)	0.55
	Tip resistance in rock	1. Canadian Geotechnical Society (1985)	0.50
		2. PMT Method (Canadian Geotechnical Society 1985)	0.50
		3. O'Neill and Reese (1999)	0.50
	Side resistance, IGMs	1. O'Neill and Reese (1999)	0.60
	Tip resistance, IGMs	1. O'Neill and Reese (1999)	0.55
	Static load test Compression, all materials		≤0.70*
Nominal Uplift Resistance of Single-Drilled Shaft	Rock	Horvath and Kenney (1979)	0.40
		Carter and Kulhawy (1988)	0.40
		Load Test	0.60

*Depends on the number of load tests and site variability.
AASHTO *LRFD Bridge Design Specifications*, 2006 Interim.

presented in the draft 2006 Interim AASHTO *LRFD Bridge Design Specifications* are not the same as those originally proposed by Horvath and Kenney (1979) and by Carter and Kulhawy (1988), but are nonetheless attributed to those studies. Furthermore, both studies have been superseded by more recent research. In future calibration studies for LRFD applications and for updates of AASHTO specifications, consideration of alternative design equations for side resistance should be considered and the most up-to-date research should be referenced.

The draft 2006 Interim AASHTO *LRFD Bridge Design Specifications* allow the use of methods other than those given in Table 20, especially if the method is "locally recognized and considered suitable for regional conditions . . . if resistance factors are developed in a manner that is consistent with the development of the resistance factors for the method(s) provided in these Specifications."

AASHTO specifies resistance factors for base resistance based on the two methods given by the Canadian Geotechnical Society (*Canadian Foundation Engineering Manual* 1985). The first is according to Eqs. 63–65 and is a straightforward method to apply, provided the rock satisfies the criteria of being horizontally jointed and the appropriate parameters can be determined. Standard logging procedures for rock core would normally provide the required information. The second method is based on PMT and is given by Eq. 66. As noted in chapter two, only a few states reported using the PMT in rock. The third method for base resistance is O'Neill and Reese (1999) and the two equations given by AASHTO correspond to Eq. 43 of this report for massive rock and Eq. 57 of this report for highly fractured rock.

AASHTO also allows higher resistance factors on both side and base resistances when they are determined from a field load test. The cost benefits achieved by using a load test as the basis for design can help to offset the costs of conducting load tests. This issue is considered further in chapter five. Finally, AASHTO recommends that all of the resistance factors given in Table 20 be reduced by 20% when used for the design of nonredundant shafts; for example, a single shaft supporting a bridge pier.

SUMMARY

The principal factors controlling the behavior of rock-socketed foundations under axial loading are identified and discussed. It is concluded from this study that sufficient tools are currently available for transportation agency personnel to design rock-socketed shafts for axial loading conditions that provide adequate load carrying capacity without being overly conservative.

The principal performance design criteria for axial loading are (1) adequate capacity and (2) ability to limit vertical deformation. Research published over the past 25 years has resulted in methods for predicting ultimate side resistance of shafts in rock that can be selected by a designer on the basis of commonly measured geomaterial properties and that account for levels of uncertainty associated with the project. For example, Eq. 30 with C taken equal to 1.0 provides a conservative estimate of side resistance for preliminary design or for final design of small structures or at sites where no additional testing is planned. If laboratory CNS testing is conducted to measure rock–concrete interface strength, higher values of side resistance can be justified for design. If field load tests are conducted, they normally result in higher side resistance values than given by Eq. 30 (with $C = 1.0$) and higher resistance factors are allowed by AASHTO for results based on load tests. If field load testing demonstrates that a particular construction technique; for example, artificial roughening the walls of a socket, can increase side resistance, then it may be possible to justify the use of Eq. 30 with values of C higher than 1.0.

Rational methods are available for estimating ultimate base resistance of rock sockets. A first order approximation based on strength of intact rock is given by Zhang and Einstein (1998) (see Figure 33). For fractured rock, a reasonable estimate can be made if the GSI (or RMR) is evaluated (see Figures 31 and 32). Although most states surveyed do not currently use GSI and RMR, the parameters required for its implementation can be obtained during the course of standard core logging procedures. The method recommended by CGS and adopted by AASHTO is applicable to moderately jointed sedimentary rocks, which is the most commonly encountered rock type for rock-socketed foundations. A method based on PMT provides another practical approach for calculating base resistance.

A source of uncertainty in rock-socket design stems from attempting to combine side and base resistances at a specific value of downward displacement; for example, at the specified limiting value of settlement or at the strength limit state. A relatively straightforward analysis based on elastic continuum theory, as given by Carter and Kulhawy (1988), is presented in the form of closed-form expressions that predict axial load-deformation and base load transfer for typical conditions encountered in practice. Similar analytical approaches for IGMs are given by O'Neill and Reese (1999). These equations are easily implemented in spreadsheet or other convenient form and allow designers to make rational estimates of load carried by both side and base at specified displacements. The survey questionnaire shows that this method is used by some state DOTs. The approach should be evaluated further against field load test measurements and, if verified, used more widely. Alternatively, the design charts given in Rowe and Armitage (1987a,b) provide a rational means of estimating axial load-displacement behavior and base load transfer. The charts are based on rigorous numerical modeling and are the benchmark against which the Carter and Kulhawy closed-form expressions were evaluated. However, the charts are more cumbersome to use. A computer program that models the full load-displacement curve, ROCKET, is available, but requires input parameters that normally are not determined by transportation agencies, such as triaxial strength properties and socket roughness parameters. However, for agencies interested in obtaining the required material properties, this program offers an effective method for axial load-deformation analysis.

Methods for calculating nominal (ultimate) unit side and base resistances and associated resistance factors according to the Interim 2006 AASHTO *LRFD Bridge Design Specifications* are summarized in Table 20. Considering the information identified by the literature review, in particular recent studies on correlation equations for unit side resistance, a suggested improvement in future specifications would be to consider design methods recommended by the more recent studies for inclusion and calibration to LRFD.

CHAPTER FOUR

DESIGN FOR LATERAL LOADING

SCOPE

Twenty-two states indicated in the survey that lateral loading considerations govern the design of drilled shafts in rock or IGM on at least some of their projects. Several states responded that lateral loading governs 100% of their designs. The survey also demonstrated that the most widely used analysis is the *p-y* method, although other methods are also used. In this chapter, analytical models identified by the literature search and by the survey are reviewed and evaluated for their applicability to rock sockets. Structural issues associated with rock-socketed shafts are considered.

DESIGN PROCESS

Deep foundations supporting bridge structures may be subjected to lateral loading from a variety of sources, including earth pressures, centrifugal forces from moving vehicles, braking forces, wind loading, flowing water, waves, ice, seismic forces, and impact. Reese (1984) describes numerous examples of bridges, overhead sign structures, and retaining structures as typical examples of transportation facilities that must sustain significant lateral loading. Drilled shafts are often selected for such structures because they can be designed to sustain lateral loading by proper sizing of the shaft and by providing a sufficient amount of reinforcing steel to resist the resulting bending moments.

Design for lateral loading of drilled shafts requires significant interaction between geotechnical and structural engineers. As described in chapter one, the Bridge Office (structural) is responsible for structural analysis and design of the superstructure and the foundations. However, to model foundation response to lateral loading it is necessary to account for soil/rock-structure interaction. The Geotechnical Division (GD) normally conducts foundation analysis using the models described in this chapter. For preliminary foundation design, geotechnical modeling of foundation response by the GD is used to provide the Bridge Office with information such as depth of fixity, trial designs (diameter and depth) of drilled shafts, and equivalent lateral spring values for use in seismic analysis of the superstructure. The Bridge Office conducts analyses of the superstructure based on models that include fixed-end columns at the depth of fixity. The structural analysis may result in revised loads that are then used by GD to perform revised lateral load analyses. In addition, the Bridge Office may conduct their own analyses using soil and rock-structure interaction models with soil and rock properties provided by GD. The Bridge Office uses the modeling results to establish design parameters for drilled shaft reinforced concrete. According to the 2006 Interim AASHTO *LRFD Bridge Design Specifications*, the strength limit state for lateral resistance of deep foundations is structural only. The basic assumption is that "failure" of the soil/rock does not occur; instead, the geomaterials continue to deform at constant or slightly increasing resistance. Failure occurs when the foundation reaches the structural limit state, defined as the loading at which the nominal combined bending and axial resistance is reached.

Axial loading in both compression and uplift requires structural analysis and reinforced-concrete design for drilled shafts. When lateral loading is not significant, structural design is reasonably straightforward. When lateral loading is significant, the combined effects of lateral and axial loading are analyzed using models described in this chapter, which account for the effect of axial load by treating the shaft as a beam column. For this reason, structural design issues associated with rock-socketed shafts are addressed in this chapter.

ROCK-SOCKETED FOUNDATIONS FOR LATERAL LOADING

Rock-socketed shafts provide significant benefits for carrying lateral loads. Embedment into rock, in most cases, reduces the lateral displacements substantially compared with a deep foundation in soil. To take full advantage of rock-socketed drilled shafts, designers must have confidence in the analytical tools used for design. The survey questionnaire shows that traditional methods of analysis for lateral loading of piles and drilled shafts in soil are the most widely used methods currently employed for rock sockets. Recent research has led to some advancements for applying these methods to rock. In addition, several researchers have proposed new analytical methods that provide designers with useful tools for predicting load-displacement response and/or structural response of the reinforced-concrete shaft. Each method has advantages and disadvantages for design purposes and these are discussed in the following sections.

ANALYTICAL METHODS

A laterally loaded deep foundation is the classic example of a soil–structure interaction problem. The soil or rock reaction depends on the foundation displacement, whereas displacement is dependent on the soil or rock response and flexural rigidity of the foundation. In most methods of analysis, the foundation is treated as an elastic beam or elastic beam-column. The primary difference in analytical methods used to date lies in the approach used to model the soil and/or rock mass response. Methods of analysis fall into two general categories: (1) subgrade reaction and (2) elastic continuum theory.

Subgrade reaction methods treat the deep foundation analytically as a beam on elastic foundation. The governing differential equation (Hetenyi 1946) is given by

$$EI\frac{d^4y}{dz^4} + P_z\frac{d^2y}{dz^2} - p - w = 0 \tag{97}$$

in which EI = flexural rigidity of the deep foundation, y = lateral deflection of the foundation at a point z along its length, P_z = axial load on the foundation, p = lateral soil/rock reaction per unit length of foundation, and w = distributed load along the length of the shaft (if any). In the most commonly used form of subgrade reaction method, the soil reaction–-displacement response is represented by a series of independent nonlinear springs described in terms of *p-y* curves (Reese 1984). A model showing the concept is provided in Figure 36. The soil or rock is replaced by a series of discrete mechanisms (nonlinear springs) so that at each depth z the soil or rock reaction p is a nonlinear function of lateral deflection y. Ideally, each *p-y* curve would represent the stress–strain and strength behavior of the soil or rock, effects of confining stress, foundation diameter and depth, position of the water table, and any other factors that determine the actual soil/rock reaction. In practice, a great deal of effort and research has been aimed at developing methods for selecting appropriate *p-y* curves. All of the proposed methods are empirical and there is no independent test to determine the relevant *p-y* curve. The principal limitations of the *p-y* method normally cited are that: (1) theoretically, the interaction of soil or rock between adjacent springs is not taken into account (no continuity), and (2) the *p-y* curves are not related directly to any measurable material properties of the soil or rock mass or of the foundation. Nevertheless, full-scale load tests and theory have led to recommendations for establishing *p-y* curves for a variety of soil types (Reese 1984). The method is attractive for design purposes because of the following:

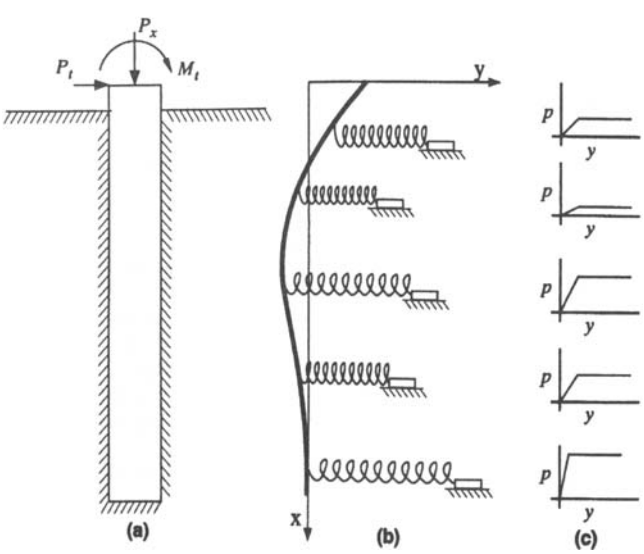


FIGURE 36 Subgrade reaction model based on *p-y* curves (Reese 1997).

- Ability to simulate nonlinear behavior of the soil or rock;
- Ability to follow the subsurface stratigraphy (layering) closely;
- Can account for the nonlinear flexural rigidity (*EI*) of a reinforced-concrete shaft;
- Incorporates realistic boundary conditions at the top of the foundation;
- Solution provides deflection, slope, shear, and moment as functions of depth;
- Solution provides information needed for structural design (shear and moment); and
- Computer solutions are readily available.

Boundary conditions that can be applied at the top of the foundation include: (1) degree of fixity against rotation or translation and (2) applied loads (moment, shear, and axial). With a given set of boundary conditions and a specified family of *p-y* curves, Eq. 97 is solved numerically, typically using a finite-difference scheme. An iterative solution is required to incorporate the nonlinear *p-y* curves as well as the nonlinear moment versus *EI* relationship (material and geometric nonlinearities) for reinforced-concrete shafts.

The elastic continuum approach for laterally loaded deep foundations was developed by Poulos (1971), initially for analysis of a single pile under lateral and moment loading at the pile head. The numerical solution is based on the boundary element method, with the pile modeled as a thin elastic strip and the soil modeled as a homogeneous, isotropic elastic material. This approach was used to approximate socketed piles by Poulos (1972) by considering two boundary conditions at the tip of the pile: (1) the pile is completely fixed against rotation and displacement at the tip (rock surface) and (2) the pile is free to rotate but fixed against translation (pinned) at the tip. The fixed pile tip condition was intended to model a socketed deep foundation, whereas the pinned tip was intended to model a pile bearing on, but not embedded into, rock. Although these tip conditions do not adequately model the behavior of many rock-socketed shafts, the analyses served to demonstrate some important aspects of socketed deep foundations. For relatively stiff foundations, which applies to many drilled shafts, considerable reduction in displacement at the pile head can be achieved by socketing, especially if the effect of the socket is to approximate a "fixed" condition at the soil/rock interface.

The elastic continuum approach was further developed by Randolph (1981) through use of the finite-element method (FEM). Solutions presented by Randolph cover a wide range of conditions for flexible piles and the results are presented in the form of charts as well as convenient closed-form solutions for a limited range of parameters. The solutions do not adequately cover the full range of parameters applicable to rock-socketed shafts used in practice. Extension of this approach by Carter and Kulhawy (1992) to rigid shafts and shafts of intermediate flexibility, as described subsequently, has led to practical analytical tools based on the continuum approach.

Sun (1994) applied elastic continuum theory to deep foundations using variational calculus to obtain the governing differential equations of the soil and pile system, based on the Vlasov model for a beam on elastic foundation. This approach was extended to rock-socketed shafts by Zhang et al. (2000), and is also described in this chapter.

p-y Method for Rock Sockets

The *p-y* method of analysis, as implemented in various computer codes, is the single most widely used method for design of drilled shafts in rock. Responses to the survey questionnaire for this study showed that 28 U.S. state transportation agencies (of 32 responding) use this method. The analytical procedure is dependent on being able to represent the response of soil and rock by an appropriate family of *p-y* curves. The only reliable way to verify *p-y* curves is through instrumented full-scale load tests. The approach that forms the basis for most of the published recommendations for *p-y* curves in soil is to instrument deep foundations with strain gages to determine the distribution of bending moment over the length of the foundation during a load test. Assuming that the bending moment can be determined reliably from strain gage measurements, the moment as a function of depth can be differentiated twice to obtain p and integrated twice to obtain y. Measured displacements at the foundation head provide a boundary condition at that location. The *p-y* curves resulting from analysis of field load tests have then been correlated empirically to soil strength and stress–strain properties determined from laboratory and in situ tests.

An alternative approach for deducing *p-y* curves from load tests is to measure the shape of the deformed foundation; for example, using slope inclinometer measurements and fitting *p-y* curves to obtain agreement with the measured displacements. This approach is described by Brown et al. (1994).

Very few lateral load tests on drilled shafts in rock, with the instrumentation necessary to back-calculate *p-y* curves, have been conducted and published to date. This lack of verification can be viewed as a limitation on use of the *p-y* method for rock-socketed drilled shafts. A single study by Reese (1997) presents the only published criteria for selection of *p-y* curves in rock. A few state DOTs have developed in-house correlations for *p-y* curves in rock.

Reese (*1997*)

Reese proposed interim criteria for *p-y* curves used for analysis of drilled shafts in rock. Reese cautions that the recommendations should be considered as preliminary because of the meager amount of load test data on which they are based. The criteria are summarized as follows. For "weak rock," defined as rock with unconfined compressive strength between 0.5 MPa and 5 MPa, the shape of the *p-y* curve, as shown in Figure 37, can be described by the following equations. For the initial linear portion of the curve

$$p = K_{ir} y \qquad \text{for } y \le y_A \qquad (98)$$

For the transitional, nonlinear portion

$$p = \frac{p_{ur}}{2}\left(\frac{y}{y_{rm}}\right)^{0.25} \qquad \text{for } y \ge y_A,\ p \le p_{ur} \qquad (99)$$

$$y_{rm} = k_{rm} B \qquad (100)$$

and when the ultimate resistance is reached

$$p = p_{ur} \qquad (101)$$

where

K_{ir} = initial slope of the curve,
p_{ur} = the rock mass ultimate resistance,
B = shaft diameter, and
k_{rm} is a constant ranging from 0.0005 to 0.00005 that serves to establish the overall stiffness of the curve.

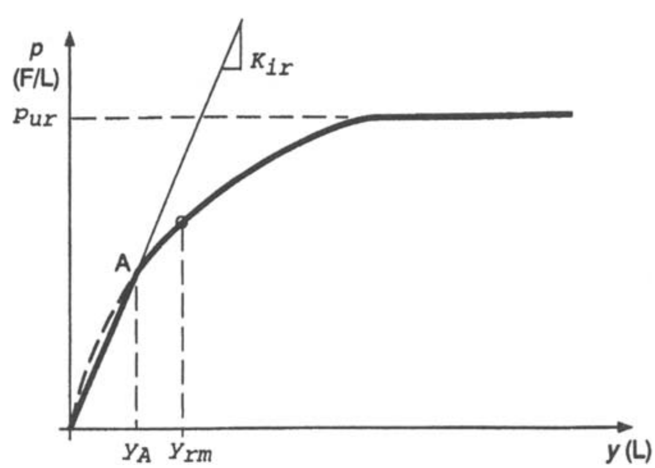


FIGURE 37 Proposed *p-y* curve for weak rock (Reese 1997).

The value of y_A corresponding to the upper limit of the initial linear portion of the curve is obtained by setting Eq. 98 equal to Eq. 99, yielding

$$y_A = \left[\frac{p_{ur}}{2\left(y_{rm}\right)^{0.25} K_{ir}}\right]^{1.333} \tag{102}$$

The following expression is recommended for calculating the rock mass ultimate resistance:

$$p_{ur} = \alpha_r q_u B\left(1 + 1.4\frac{x_r}{B}\right) \quad \text{for } 0 \le x_r \le 3B \tag{103}$$

$$p_{ur} = 5.2\alpha_r q_u B \quad \text{for } x_r \ge 3B \tag{104}$$

in which q_u = uniaxial compressive strength of intact rock, α_r = strength reduction factor, and x_r = depth below rock surface. Selection of α_r is based on the assumption that fracturing will occur at the surface of the rock under small deflections, thus reducing the rock mass compressive strength. The value of α_r is assumed to be one-third for RQD of 100 and to increase linearly to unity at RQD of zero. The underlying assumption is that, if the rock mass is already highly fractured, then no additional fracturing with accompanying strength loss will occur. However, this approach appears to have a fundamental shortcoming in that it relies on the compressive strength of the *intact* rock and not the strength of the rock mass. For a highly fractured rock mass (low RQD) with a high-intact rock strength, it seems that the rock mass strength could be overestimated.

The initial slope of the *p-y* curve, K_{ir}, is related to the initial elastic modulus of the rock mass as follows:

$$K_{ir} \cong k_{ir} E_{ir} \tag{105}$$

where E_{ir} = rock mass initial elastic modulus and k_{ir} = dimensionless constant given by

$$k_{ir} = \left(100 + \frac{400x_r}{3B}\right) \quad \text{for } 0 \le x_r \le 3B \tag{106}$$

$$k_{ir} = 500 \quad \text{for } x_r > 3B \tag{107}$$

The expressions for k_{ir} were determined by fitting a *p-y* analysis to the results of a field load test (back-fitting) in which the initial rock mass modulus value was determined from PMTs. The method recommended by Reese (1997) is to establish E_{ir} from the initial slope of a pressuremeter curve. Alternatively, Reese suggests the correlation given by Bieniawski (1978) between rock mass modulus, modulus of intact rock core, and RQD, given as expression 2 in Table 12 (chapter two) of this report. According to Reese (1997), rock mass modulus E_M determined this way is assumed to be equivalent to E_{ir} in Eq. 105.

Results of load tests at two sites are used by Reese (1997) to fit *p-y* curves according to the criteria given previously for weak rock. The first load test was located at Islamorada, Florida. A drilled shaft, 1.2 m in diameter and 15.2 m long, was socketed 13.3 m into a brittle, vuggy coral limestone. A layer of sand over the rock was retained by a steel casing and lateral load was applied 3.51 m above the rock surface. The following values were used in the equations for calculating the *p-y* curves: q_u = 3.45 MPa, α_r = 1.0, E_{ir} = 7,240 MPa, k_{rm} = 0.0005, B = 1.22 m, L = 15.2 m, and $EI = 3.73 \times 10^6$ kN-m^2. Comparison of pile head deflections measured during the load test and from *p-y* analyses are shown in Figure 38. With a constant value of *EI* as given above, the analytical results show close agreement with the measured displacements up to a lateral load of about 350 kN. By reducing the values of flexural rigidity in portions of the shaft subject to high moments, the *p-y* analysis was adjusted to yield deflections that agreed with the measured values at loads higher than 350 kN. The value of k_{rm} = 0.0005 was also determined on the basis of establishing agreement between the measured and predicted displacements.

The second case analyzed by Reese (1997) is a lateral load test conducted on a drilled shaft socketed into sandstone at a site near San Francisco. The shaft was 2.25 m in diameter with a socket length of 13.8 m. Rock mass strength and modulus values were estimated from PMT results. Three zones of rock were identified and average values of strength and modulus were assigned to each zone. The sandstone is described as "medium-to-fine-grained, well sorted, thinly bedded, very intensely to moderately fractured." Twenty values of RQD were reported, ranging from zero to 80, with an average of 45. For calculating *p-y* curves, the strength reduction factor α_r was taken as unity, on the assumption that there was "little chance of brittle fracture." Values of the other parameters

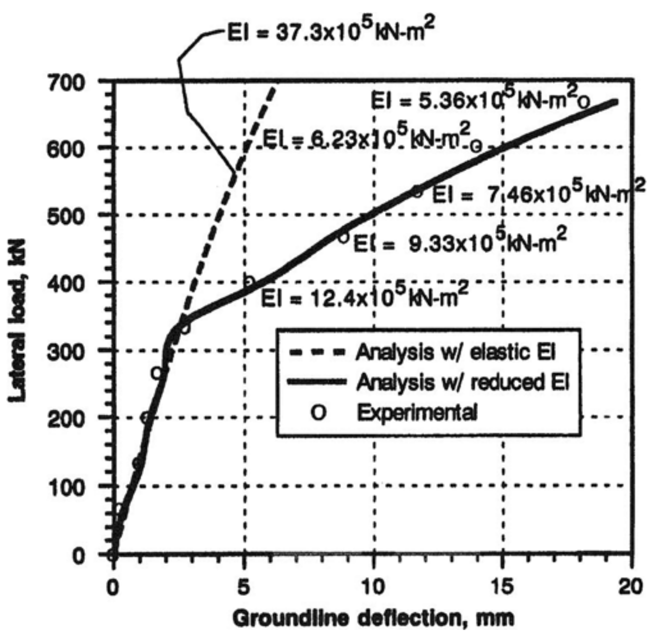


FIGURE 38 Measured and analytical deflection curves for shaft in vuggy limestone (Reese 1997).

used for p-y curve development were: $k_{rm} = 0.00005$; q_u = 1.86 MPa for depth of 0–3.9 m, 6.45 MPa for depth of 3.9–8.8 m, and 16.0 MPa for depth of more than 8.8 m; $E_{ir} = 10q_u$ (MPa) for each layer, B = 2.25 m, and $EI = 35.15 \times 10^3$ MN-m^2. The value of k_{rm} was adjusted to provide agreement between displacements given by the p-y method of analysis and measured displacements from the load test.

Figure 39 shows a comparison of the measured load-displacement curve with results produced by the p-y method of analysis, for various methods of computing the flexural rigidity (EI) of the test shaft. Methods that account for the nonlinear relationship between bending moment and EI provide a better fit than p-y analysis with a constant vale of EI. The curve labeled "Analytical" in Figure 39 was obtained using an analytical procedure described by Reese to incorporate the nonlinear moment–EI relationships directly into the numerical solution of Eq. 97, whereas the curve labeled "ACI" incorporates recommendations by the American Concrete Institute for treating the nonlinear moment–EI behavior.

Fitting of p-y curves to the results of the two load tests as described previously forms the basis for recommendations that have been incorporated into the most widely used computer programs being used by state DOTs for analysis of laterally loaded rock-socketed foundations. The program COM624 (Wang and Reese 1991) and its commercial version, LPILE (Ensoft, Inc. 2004), allow the user to assign a limited number of soil or rock types to each subsurface layer. One of the options is "weak rock." If this geomaterial selection is made, additional required input parameters are unit weight, modulus, uniaxial compressive strength, RQD, and k_{rm}. The program then generates p-y curves using Eqs. 98–107. The program documentation recommends assigning "weak rock" to geomaterials with uniaxial compressive strengths in the range of 0.5–5 MPa. The user assigns a value to k_{rm}. The documentation (Ensoft, Inc. 2004) recommends to:

> . . . examine the stress–strain curve of the rock sample. Typically, the k_{rm} is taken as the strain at 50% of the maximum strength of the core sample. Because limited experimental data are available for weak rock during the derivation of the p-y criteria, the k_{rm} from a particular site may not be in the range between 0.0005 and 0.00005. For such cases, you may use the upper bound value (0.0005) to get a larger value of y_{rm}, which in turn will provide a more conservative result.

The criteria recommended for p-y curves in the LPILE^{PLUS} users manual (Ensoft Inc. 2004) for "strong rock" is illustrated in Figure 40. Strong rock is defined by a uniaxial strength of intact rock $q_u \geq 6.9$ MPa. In Figure 40, s_u is defined as one-half of q_u and b is the shaft diameter. The p-y curve is bilinear, with the break in slope occurring at a deflection y corresponding to 0.04% of the shaft diameter. Resistance (p) is a function of intact rock strength for both portions of the curve. The criterion does not account explicitly for rock mass properties, which would appear to limit its applicability to massive rock. The authors recommend verification by load testing if deflections exceed 0.04% of the shaft diameter, which would exceed service limit state criteria in most practical situations. Brittle fracture of the rock is assumed if the resistance p becomes greater than the shaft diameter times one-half of the uniaxial compressive strength of the rock. The deflection y corresponding to brittle fracture can be determined from the diagram as 0.0024 times the shaft diameter. This level of displacement would be exceeded in many practical situations. It is concluded that the recommended criteria applies only for very small lateral deflections and is not valid for jointed rock masses. Some practitioners apply the weak rock criteria, regardless of material strength, to avoid the limitations cited earlier. The authors state that the p-y curve shown in Figure 40 "should be employed with

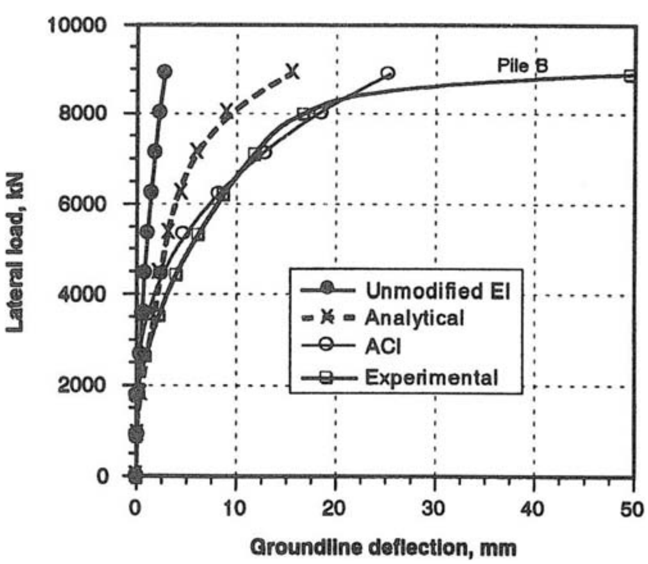


FIGURE 39 Measured and analytical deflection curves, socket in sandstone (Reese 1997).

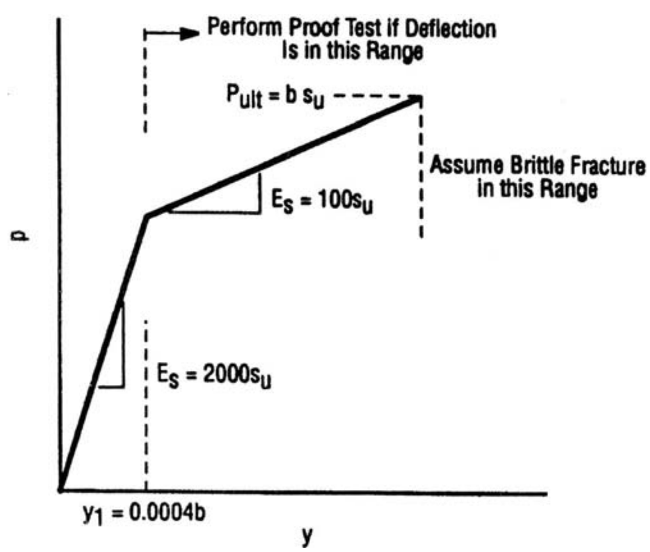


FIGURE 40 Recommended p-y curve for strong rock (Ensoft, Inc. 2004).

caution because of the limited amount of experimental data and because of the great variability in rock."

The survey questionnaire for this study found that 28 agencies use either COM624 or LPILE^{PLUS} for analysis of rock-socketed shafts under lateral loading.

The following observations are based on a review of the literature:

- Existing published criteria for *p-y* curves in rock are based on a very limited number (two) of full-scale field load tests,
- Recommendations for selecting values of input parameters required by the published criteria are vague and unsubstantiated by broad experience, and
- The *p-y* method of analysis is being used extensively despite these sources of uncertainty.

It is therefore concluded that research is needed and should be undertaken with the objective of developing improved criteria for *p-y* curves in rock. The research should include full-scale field load tests on instrumented shafts, much in the same way that earlier studies focused on the same purpose for deep foundations in soil. The *p-y* curve parameters should be related to rock mass engineering properties that can be determined by state transportation agencies using available site and material characterization methods, as described in chapter two.

Current Research by State Agencies

The literature review and the survey identified two state transportation agencies (North Carolina and Ohio) with research in progress aimed at improving the methodology for constructing *p-y* curves for weathered rock. The North Carolina study is described in a draft report by Gabr et al. (2002) and the Ohio study is summarized in a paper that was under review at the time of this writing, by Liang and Yang (2006). Both studies present recommendations for *p-y* curves based on a hyperbolic function. Two parameters are required to characterize a hyperbola, the initial tangent slope and the asymptote. For the proposed hyperbolic *p-y* models, these correspond to the subgrade modulus (K_h) and the ultimate resistance (p_{ult}), as shown in Figure 41. The hyperbolic *p-y* relationship is then given as

$$p = \frac{y}{\dfrac{1}{K_h} + \dfrac{y}{p_{ult}}} \tag{108}$$

A summary of the two studies, including recommendations for selection of the required parameters (K_h and p_{ult}), is presented.

In the North Carolina study, results of six full-scale field load tests, at three different sites, were used to develop the model. Tests were performed on shafts in Piedmont weathered profiles of sandstone, mica schist, and crystalline rock. Finite-element modeling was used to calibrate a *p-y* curve model incorporating subgrade modulus as determined from PMT readings and providing close agreement with strains and deflections measured in the load tests. The model was then used to make forward predictions of lateral load response for subsequent load tests on socketed shafts at two locations in weathered rock profiles different than those used to develop the model.

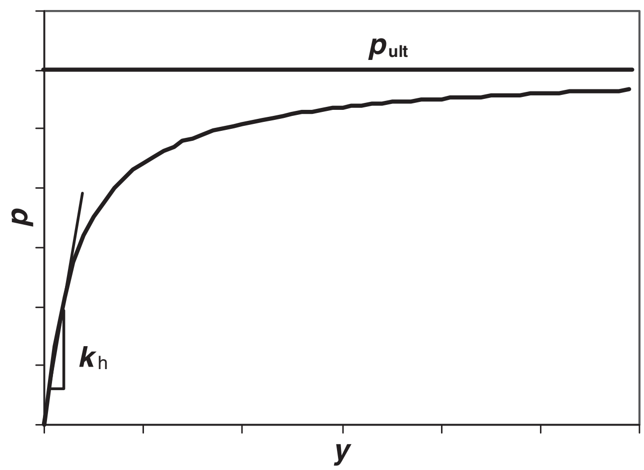


FIGURE 41 Hyperbolic *p-y* curve.

The procedure for establishing values of subgrade modulus K_h involves determination of the rock mass modulus (E_M) from PMT measurements. The coefficient of subgrade reaction is then given by:

$$k_h = \frac{0.65E_M}{B\left(1-v_r^2\right)}\left[\frac{E_M B^4}{E_s I_s}\right]^{1/12} \qquad \left(units: \frac{F}{l^3}\right) \tag{109}$$

in which B = shaft diameter, E_M = rock mass modulus, v_r = Poisson's ratio of the rock, and E_s and I_s are modulus and moment of inertia of the shaft, respectively. A procedure is given by Gabr et al. (2002) for establishing the point of rotation of the shaft. For *p-y* curves above the point of rotation, subgrade modulus is equal to the coefficient of subgrade reaction times the shaft diameter or

$$K_h = k_h B \tag{110}$$

For depths below the point of rotation, a stiffer lateral subgrade reaction is assigned and the reader is referred to Gabr et al. (2002) for the equations. An alternative procedure is presented for cases where rock mass modulus is determined using the empirical correlation given by Hoek and Brown (1997) and presented previously as expression 7 in Table 12 of chapter two. In that expression, rock mass modulus is correlated with GSI and uniaxial compressive strength of intact rock (q_u).

The second required hyperbolic curve parameter is the asymptote of the *p-y* curve, which is the ultimate resistance p_{ult}. The proposed expression is given by

$$p_{ult} = (p_L + \tau_{max})B \tag{111}$$

where p_L = limit normal stress and τ_{max} = shearing resistance along the side of the shaft. Gabr et al. adopted the following recommendation of Zhang et al. (2000) for unit side resistance:

$$\tau_{max} = 0.20\sqrt{q_u(MPa)} \tag{112}$$

The limit normal stress is estimated on the basis of Hoek–Brown strength parameters as determined through correlations with RMR and GSI, and is given by

$$p_L = \gamma' z + q_u \left(m_b \frac{\gamma' z}{q_u} + s \right)^a \tag{113}$$

in which γ' = effective unit weight of the rock mass, z = depth from the rock mass surface, and the coefficients m_b, s, and a are the Hoek–Brown coefficients given by Eqs. 12–15 in chapter two.

Results of one of the field load tests conducted for the purpose of evaluating the predictive capability of the proposed weak rock (WR) model is shown in Figure 42. The analyses were carried out using the program LPILE. Analyses were also conducted using *p-y* curves as proposed by Reese (1997), described previously, as well as several other *p-y* curve recommendations. The proposed model based on hyperbolic *p-y* curves derived from PMT measurements (labeled dilatometer in Figure 42) shows good agreement with the test results. The authors (Gabr et al. 2002) attributed the underpredicted displacements obtained using the Reese criteria to the large values of the factor k_{ir} predicted by Eqs. 106 and 107. However, the analysis did not incorporate the nonlinear moment–*EI* behavior of the concrete shaft, which reduces the predicted deflections more significantly than the *p-y* criteria. One of the limitations of the *p-y* criterion proposed by Gabr et al. (2002) is that it is based on analyses in which *EI* is taken as a constant. For proper analysis of soil–rock–structure interaction during lateral loading, the nonlinear moment–*EI* relationship should be modeled correctly.

The North Carolina DOT also reports using the program LTBASE, which analyzes the lateral load-displacement response of deep foundations as described by Gabr and Borden (1988). The analysis is based on the *p-y* method, but also accounts for base resistance by including a vertical resistance component mobilized by shaft rotation and horizontal shear resistance, as illustrated in Figure 43. Base resistance becomes significant as the relative rigidity of the shaft increases and as the slenderness ratio decreases. For relatively rigid rock sockets, mobilization of vertical and shear resistance at the tip could increase overall lateral capacity significantly, and base resistance effects should be considered. Gabr et al. (2002) stated that the hyperbolic WR *p-y* curve model is now incorporated into LTBASE, but no results were given.

In the Ohio DOT study, Liang and Yang (2006) also propose a hyperbolic *p-y* curve criterion. The derivation is based on theoretical considerations and finite-element analyses. Results of two full-scale, fully instrumented field load tests are compared with predictions based on the proposed *p-y* curve criterion. The initial slope of the hyperbolic *p-y* curve is given by the following semi-empirical equation:

$$K_h = E_M \left(\frac{B}{B_{ref}} \right) e^{-2\nu_r} \left(\frac{E_s I_s}{E_M B^4} \right)^{0.284} \tag{114}$$

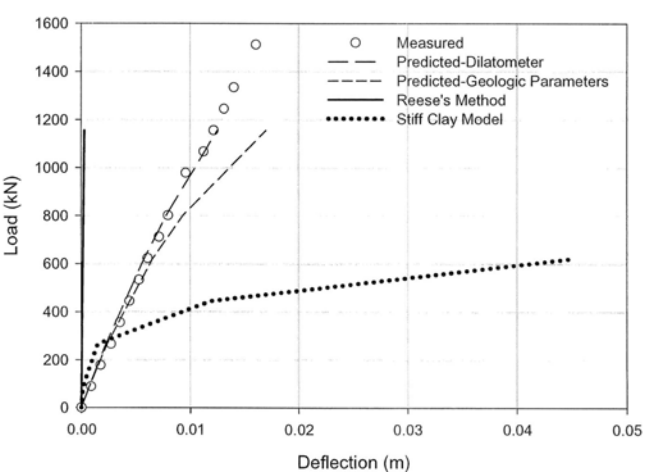


FIGURE 42 Measured lateral load deflection versus predicted (Gabr et al. 2002).

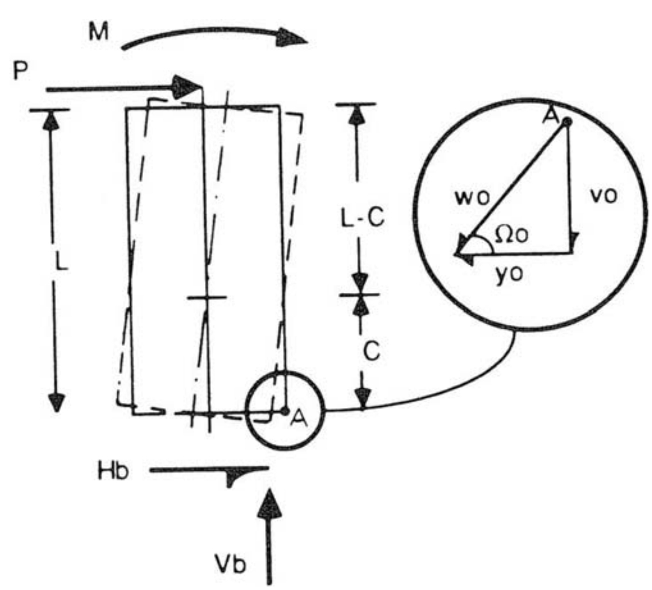


FIGURE 43 Base deformation as a function of shaft rotation (Gabr and Borden 1988).

in which B_{ref} = a reference diameter of 0.305 m (1 ft) and all other terms are as defined above for Eq. 109. Liang and Yang (2006) recommend modulus values E_M from PMTs for use in Eq. 114, but in the absence of PMT measurements they present the following correlation equation relating E_M to modulus of intact rock and GSI:

$$E_M = \frac{E_r}{100} e^{\frac{\mathrm{GSI}}{21.7}} \tag{115}$$

where E_r = elastic modulus of intact rock obtained during uniaxal compression testing of core samples. Eq. 115 is also expression 8 of Table 12 in chapter two. Liang and Yang (2006) present two equations for evaluating p_{ult}. The first corresponds to a wedge failure mode, which applies to rock mass near the ground surface. The second applies to rock mass at depth and is given by

$$p_{ult} = \left(\frac{\pi}{4} p_L + \frac{2}{3} \tau_{\max} - p_A \right) B \tag{116}$$

where p_L = limit normal stress, $\tau_{\max}$ = shearing resistance along the side of the shaft, and p_A = horizontal active pressure. Eq. 116 is similar to Eq. 111 (Gabr et al. 2002), but accounts for active earth pressure acting on the shaft. Both methods incorporate the Hoek–Brown strength criterion for rock mass to evaluate the limit normal stress p_L, and both rely on correlations with GSI to determine the required Hoek–Brown strength parameters. In the Liang and Yang (2006) approach, p_{ult} at each depth is taken as the smaller of the two values obtained from the wedge analysis or by Eq. 116.

A source of uncertainty in all of the proposed *p-y* criteria derives from the choice of method for selecting rock mass modulus when more than one option is available. For example, using the pressuremeter and GSI data reported by Gabr et al. (2002) significantly different values of modulus are obtained for the same site. In some cases, the measured shaft load-displacement response (from load testing) shows better agreement with *p-y* curves developed from PMT modulus, whereas another load test shows better agreement with *p-y* curves developed from GSI-derived modulus. Proper selection of rock mass modulus for foundation design is one of the challenges for design of rock-socketed shafts, as pointed out in chapter two. This issue becomes most important when *p-y* curves for lateral load analysis are based on rock mass modulus. Both the Reese (1997) criteria and the hyperbolic criteria require rock mass modulus to determine the slope of *p-y* curves.

The North Carolina and Ohio programs provide examples of state DOT efforts to advance the state of practice in design of rock-socketed foundations. The programs incorporate careful site investigations using available methods for characterizing rock mass engineering properties (RMR, GSI) as well as in situ testing (PMT). Both programs are based on analysis of full-scale field load tests on instrumented shafts. However, the proposed equations for generating *p-y* curves differ between the two proposed criteria and both models will result in different load-displacement curves. It is not clear if either model is applicable to rock sockets other than those used in its development. Both sets of load tests add to the database of documented load tests now available to researchers. A useful exercise would be to evaluate the North Carolina proposed criteria against the Ohio load test results and vice versa.

Florida Pier

Several states reported using other computer programs that are based on the *p*-y method of analysis. Seven agencies report using the Florida Bridge Pier Analysis Program (FBPIER) for analysis of rock-socketed shafts. Of those seven, six also report using COM624 and/or LPILE. The FBPIER, described by Hoit et al. (1997), is a nonlinear, finite-element analysis program designed for analyzing bridge pier substructures composed of pier columns and a pier cap supported on a pile cap and piles or shafts including the soil (or rock). FBPIER was developed to provide an analytical tool allowing the entire pier structure of a bridge to be analyzed at one time, instead of multiple iterations between foundation analysis programs (e.g., COM624) and structural analysis programs. Basically, the structural elements (pier column, cap, pile cap, and piles) are modeled using standard structural finite-element analysis, including nonlinear capabilities (nonlinear *M–EI* behavior), whereas the soil response is modeled by nonlinear springs (Figure 44). Axial soil response is modeled in terms of *t-z* curves, whereas lateral response is modeled in terms of *p-y* curves. The program has built-in criteria for *p-y* curves in soil, based on published recommendations and essentially similar to those employed in LPILE. User-defined *p-y* curves can also be specified. To simulate rock, users currently apply the criteria for either soft clay (Matlock 1970) or stiff clay (Reese and Welch 1975) but with strength and stiffness properties of the rock, or user-defined curves are input. Research is underway to incorporate improved *p-y* curve criteria into FBPIER, specifically for

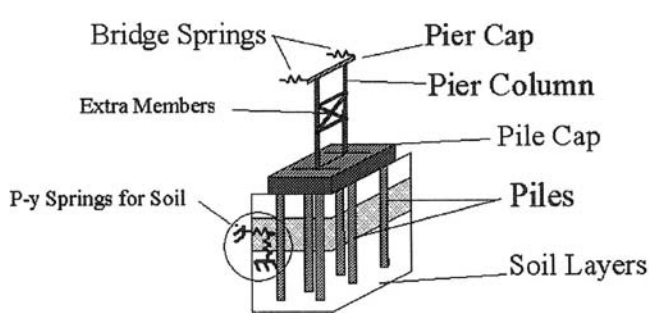


FIGURE 44 Florida pier model for structure and foundation elements (Hoit et al. 1997).

Florida limestone, as described by McVay and Niraula (2004). Centrifuge tests were conducted in which instrumented model shafts embedded in a synthetic rock (to simulate Florida limestone) were subjected to lateral loading. Strain gage measurements were used to back-calculate *p-y* curves, which are presented in normalized form, with *p* normalized by shaft diameter and rock compressive strength (p/Bq_u) and *y* normalized by shaft diameter (y/B). There is no analytical expression recommended for new *p*-y curve criteria and the report recommends that field testing be undertaken on full-size drilled shafts to validate the derived *p-y* curves established from the centrifuge tests before they are employed in practice.

Strain Wedge Model

The strain wedge (SW) model has been applied to laterally loaded piles in soil, as described by Ashour et al. (1998). The 2006 Interim AASHTO *LRFD Bridge Design Specifications* identify the SW model as an acceptable method for lateral load analysis of deep foundations. The 3-D soil–pile interaction behavior is modeled by considering the lateral resistance that develops in front of a mobilized passive wedge of soil at each depth. Based on the soil stress–strain and strength properties, as determined from laboratory triaxial tests, the horizontal soil strain (ε) in the developing passive wedge in front of the pile is related to the deflection pattern (y) versus depth. The horizontal stress change ($\Delta\sigma_H$) in the developing passive wedge is related to the soil–pile reaction (p), and the nonlinear soil modulus is related to the nonlinear modulus of subgrade reaction, which is the slope of the *p-y* curve. The SW model can be used to develop *p-y* curves for soil that show good agreement with load test results (Ashour and Norris 2000). Theoretically, the SW model overcomes some of the limitations of strictly empirically derived *p-y* curves because the soil reaction (p) at any given depth depends on the response of the neighboring soil layers (continuity) and properties of the pile (shape, stiffness, and head conditions). Ashour et al. (2001) proposed new criteria for *p-y* curves in weathered rock for use with the SW model. The criteria are described by the authors as being based on the weak rock criteria of Reese (1997) as given by Eqs. 98-104, but modified to account for the nonlinear rock mass modulus and the strength of the rock mass in terms of Mohr–Coulomb strength parameters c and ϕ. Ashour et al. (2001) reported good agreement between the SW analysis and a field load test reported by Brown (1994). One state DOT (Washington) reports using the computer program (S-Shaft) based on the SW model that incorporates the *p-y* curve criteria for weathered rock. However, the program has not yet been used for design of a socketed shaft (J. Cuthbertson, personal communication, Sep. 30, 2005). The SW model and proposed *p-y* criteria of Ashour et al. (2001) warrant further consideration and should be evaluated against additional field load test results (e.g., the tests reported by Gabr et al. 2002 and Liang and Yang 2006).

Continuum Models for Laterally Loaded Sockets

Carter and Kulhawy (1992)

Carter and Kulhawy (1988, 1992) studied the behavior of flexible and rigid shafts socketed into rock and subjected to lateral loads and moments. Solutions for the load-displacement relations were first generated using finite-element analyses. The finite-element analyses followed the approach of Randolph (1981) for flexible piles under lateral loading. Based on the FEM solutions, approximate closed-form equations were developed to describe the response for a range of rock-socket parameters typically encountered in practice. The results provide a first-order approximation of horizontal groundline displacements and rotations and can incorporate an overlying soil layer. The method is summarized as follows.

Initially, consider the case where the top of the shaft corresponds to the top of the rock layer (Figure 45). The shaft is idealized as a cylindrical elastic inclusion with an effective Young's modulus (E_e), Poisson's ratio (v_c), depth (D), and diameter (B), subjected to a known lateral force (H), and an overturning moment (M). For a reinforced-concrete shaft having an actual flexural rigidity equal to $(EI)_c$, the effective Young's modulus is given by

$$E_e = \frac{(EI)_c}{\frac{\pi B^4}{64}} \tag{117}$$

It is assumed that the elastic shaft is embedded in a homogeneous, isotropic elastic rock mass, with properties E_r and v_r. Effects of variations in the Poisson's ratio of the rock mass (v_r), are represented approximately by an equivalent shear modulus of the rock mass (G^*), defined as:

$$G^* = G_r\left(1 + \frac{3v_r}{4}\right) \tag{118}$$

in which G_r = shear modulus of the elastic rock mass. For an isotropic rock mass, the shear modulus is related to E_r and v_r by

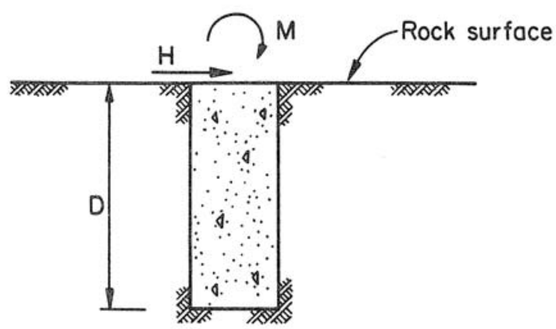


FIGURE 45 Lateral loading of rock-socketed shaft (Carter and Kulhawy 1992).

$$G_r = \frac{E_r}{2(1+v_r)} \tag{119}$$

Based on a parametric study using finite-element analysis, it was found that closed-form expressions could be obtained to provide reasonably accurate predictions of horizontal displacement (u) and rotation (θ) at the head of the shaft for two limiting cases. The two cases correspond to flexible shafts and rigid shafts. The criterion for a flexible shaft is

$$\frac{D}{B} \geq \left(\frac{E_e}{G^*}\right)^{2/7} \tag{120}$$

For shafts satisfying Eq. 120, the response depends only on the modulus ratio (E_e/G^*) and Poisson's ratio of the rock mass (v_r) and is effectively independent of D/B. The following closed-form expressions, suggested by Randolph (1981), provide accurate approximations for the deformations of flexible shafts:

$$u = 0.50\left(\frac{H}{G^*B}\right)\left(\frac{E_e}{G^*}\right)^{-1/7} + 1.08\left(\frac{M}{G^*B^2}\right)\left(\frac{E_e}{G^*}\right)^{-3/7} \tag{121}$$

$$\theta = 1.08\left(\frac{H}{G^*B^2}\right)\left(\frac{E_e}{G^*}\right)^{-3/7} + 6.40\left(\frac{M}{G^*B^3}\right)\left(\frac{E_e}{G^*}\right)^{-5/7} \tag{122}$$

in which u = groundline deflection and θ = groundline rotation of the shaft.

Carter and Kulhawy (1992) reported that the accuracy of the above equations is verified for the following ranges of parameters: $1 \leq E_e/E_r \leq 10^6$ and $D/B \geq 1$.

The criterion for a rigid shaft is

$$\frac{D}{B} \leq 0.05\left(\frac{E_e}{G^*}\right)^{1/2} \tag{123}$$

and

$$\frac{E_e/G^*}{\left(B/2D\right)^2} \geq 100 \tag{124}$$

When Eqs. 123 and 124 are satisfied, the displacements of the shaft will be independent of the modulus ratio (E_e/E_r) and will depend only on the slenderness ratio (D/B) and Poisson's ratio of the rock mass (v_r). The following closed-form expressions give reasonably accurate displacements for rigid shafts:

$$u = 0.4\left(\frac{H}{G^*B}\right)\left(\frac{2D}{B}\right)^{-1/3} + 0.3\left(\frac{M}{G^*B^2}\right)\left(\frac{2D}{B}\right)^{-7/8} \tag{125}$$

$$\theta = 0.3\left(\frac{H}{G^*B^2}\right)\left(\frac{2D}{B}\right)^{-7/8} + 0.8\left(\frac{M}{G^*B^3}\right)\left(\frac{2D}{B}\right)^{-5/3} \tag{126}$$

The accuracy of Eqs. 125 and 126 has been verified for the following ranges of parameters: $1 \leq D/B \leq 10$ and $E_e/E_r \geq 1$.

Shafts can be described as having intermediate stiffness whenever the slenderness ratio is bounded approximately as follows:

$$0.05\left(\frac{E_e}{G^*}\right)^{1/2} < \frac{D}{B} < \left(\frac{E_e}{G^*}\right)^{2/7} \tag{127}$$

For the intermediate case, Carter and Kulhawy suggested that the displacements be taken as 1.25 times the maximum of either (1) the predicted displacement of a rigid shaft with the same slenderness ratio (D/B) as the actual shaft or (2) the predicted displacement of a flexible shaft with the same modulus ratio (E_e/G^*) as the actual shaft. Values calculated in this way should, in most cases, be slightly larger than those given by the more rigorous finite-element analysis for a shaft of intermediate stiffness.

Carter and Kulhawy next considered a layer of soil of thickness D_s overlying rock, as shown in Figure 46. The analysis is approached by structural decomposition of the shaft and its loading, as shown in Figure 46*b*. It was assumed that the magnitude of applied lateral loading is sufficient to cause yielding within the soil from the ground surface to the top of the rock mass. The portion of the shaft within the soil is then analyzed as a determinant beam subjected to known loading. The displacement and rotation of point *A* relative to point *O* can be determined by established techniques of structural analysis. The horizontal shear force (H_o) and bending moment (M_o) acting in the shaft at the rock surface level can be computed from statics, and the displacement and rotation at this level can be computed by the methods described previously. The overall groundline displacements can then be calculated by superposition of the appropriate parts.

Determination of the limiting soil reactions is recommended for the two limiting cases of cohesive soil in undrained loading ($\phi = 0$) and frictional soil ($c = 0$) in drained loading. Ultimate resistance for shafts in cohesive soils is based on the method of Broms (1964a), in which the undrained

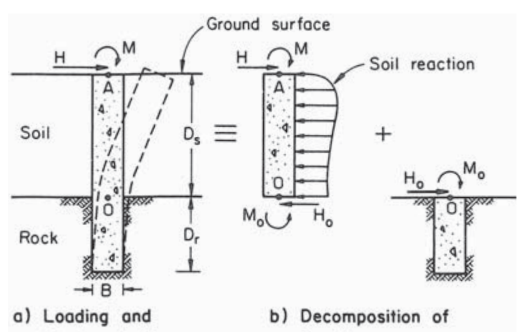


FIGURE 46 Rock-socketed shaft with overlying layer (Carter and Kulhawy 1992).

soil resistance ranges from zero at the ground surface to a depth of 1.5B and has a constant value of 9s_u below this depth, where s_u = soil undrained shear strength. For socketed shafts extending through a cohesionless soil layer, the following limiting pressure suggested by Broms (1964b) is assumed:

$$p_u = 3K_p\sigma'_v \tag{128}$$

$$K_p = \frac{1+\sin\phi'}{1-\sin\phi'} \tag{129}$$

in which σ'_v = vertical effective stress and ϕ' = effective stress friction angle of the soil. For both cases (undrained and drained) solutions are given by Carter and Kulhawy (1992) for the displacement, rotation, shear, and moment at point O of Figure 46. The contribution to groundline displacement and rotation from the loading transmitted to the rock mass (H_o and M_o) is determined based on Eqs. 121 and 122 or Eqs. 125 and 126 and added to the calculated displacement and rotation at the top of the socket to determine overall groundline response.

Application of the proposed theory is described by Carter and Kulhawy (1992) through back-analysis of a single case involving field loading of a pair of rock-socketed shafts. The method has not been evaluated against a sufficient database of field performance, and further research is needed to assess its reliability. The analysis was developed primarily for application to electrical transmission line foundations in rock, although the concepts are not limited to foundations supporting a specific type of structure. The approach is attractive for design purposes, because the closed-form equations can be executed by hand or on a spreadsheet.

Carter and Kulhawy (1992) stated that the assumption of yield everywhere in the soil layer may represent an oversimplification, but that the resulting predictions of groundline displacements will overestimate the true displacements, giving a conservative approximation. However, the assumption that the limit soil reaction is always fully mobilized may lead to erroneous results by overestimating the load carried by the soil and thus underestimating the load transmitted to the socket. Furthermore, groundline displacements may be underestimated because actual soil resistance may be smaller than the limiting values assumed in the analysis.

Zhang et al. (2000)

Zhang et al. (2000) extended the continuum approach to predict the nonlinear lateral load-displacement response of rock-socketed shafts. The method considers subsurface profiles consisting of a soil layer overlying a rock layer. The deformation modulus of the soil is assumed to vary linearly with depth, whereas the deformation modulus of the rock mass is assumed to vary linearly with depth and then to stay constant below the shaft tip. Effects of soil and/or rock mass yielding on response of the shaft are considered by assuming that the soil and/or rock mass behaves linearly elastically at small strain levels and yields when the soil and/or rock mass reaction force p (force/length) exceeds the ultimate resistance p_{ult} (force/length).

Analysis of the loaded shaft as an elastic continuum is accomplished using the method developed by Sun (1994). The numerical solution is by a finite-difference scheme and incorporates the linear variation in soil modulus and linear variation in rock mass modulus above the base of the shaft. Solutions obtained for purely elastic responses are compared with those of Poulos (1971) and finite-element solutions by Verruijt and Kooijman (1989) and Randolph (1981). Reasonable agreement with those published solutions is offered as verification of the theory, for elastic response.

The method is extended to nonlinear response by accounting for local yielding of the soil and rock mass. The soil and rock mass are modeled as elastic, perfectly plastic materials, and the analysis consists of the following steps:

1. For the applied lateral load H and moment M, the shaft is analyzed by assuming the soil and rock mass are elastic, and the lateral reaction force p of the soil and rock mass along the shaft is determined by solution of the governing differential equation and boundary conditions at the head of the shaft.
2. The computed lateral reaction force p is compared with the ultimate resistance p_{ult}. If $p > p_{ult}$, the depth of yield z_y in the soil and/or rock mass is determined.
3. The portion of the shaft in the unyielded soil and/or rock mass ($z_y \le z \le L$) is considered to be a new shaft and analyzed by ignoring the effect of the soil and/or rock mass above the level $z = z_y$. The lateral load and moment at the new shaft head are given by:

$$H_o = H - \int_0^{z_y} p_{ult}dz \tag{130}$$

$$M_o = M + Hz_y - \int_0^{z_y} p_{ult}\left(z_y - z\right)dz \tag{131}$$

4. Steps 2 and 3 are repeated and the iteration is continued until no further yielding of soil or rock mass occurs.
5. The final results are obtained by decomposition of the shaft into two parts, which are analyzed separately, as illustrated previously in Figure 46. The section of the shaft in the zone of yielded soil and/or rock mass is analyzed as a beam subjected to a distributed load of magnitude p_{ult}. The length of shaft in the unyielded zone of soil and/or rock mass is analyzed as a shaft with the soil and/or rock mass behaving elastically.

Ultimate resistance developed in the overlying soil layer is evaluated for the two conditions of undrained loading (ϕ = 0) and fully drained loading (c = 0). For fine-grained soils (clay), undrained loading conditions are assumed and the limit pressure is given by

$$p_{ult} = N_p c_u B \quad (132)$$

$$N_p = 3 + \frac{\gamma'}{c_u} z + \frac{J}{2R} z \leq 9 \quad (133)$$

in which c_u = undrained shear strength, B = shaft diameter, γ' = average effective unit weight of soil above depth z, J = a coefficient ranging from 0.25 to 0.5, and R = shaft radius. For shafts in sand, a method attributed to Fleming et al. (1992) is given as follows:

$$p_{ult} = K_p^2 \gamma' z B \quad (134)$$

where K_p = Rankine coefficient of passive earth pressure defined by Eq. 129. Ultimate resistance of the rock mass is given by

$$p_{ult} = \left(p_L + \tau_{\max}\right) B \quad (135)$$

where τ_{max} = maximum shearing resistance along the sides of the shaft (e.g., Eq. 30 of chapter three) and p_L = normal limit resistance. The limit normal stress p_L is evaluated using the Hoek–Brown strength criterion with the strength parameters determined on the basis of correlations to GSI. The resulting expression was given previously as Eq. 113.

According to Zhang et al. (2000), a computer program was written to execute this procedure. Predictions using the proposed method are compared with results of field load tests reported by Frantzen and Stratten (1987) for shafts socketed into sandy shale and sandstone. Computed pile head deflections show reasonable agreement with the load test results. The method appears to have potential as a useful tool for foundations designers. Availability of the computer program is unknown. Programming the method using a finite-difference scheme as described by Zhang et al. (2000) is also possible.

Discontinuum Models

A potential mode of failure for a laterally loaded shaft in rock is by shear failure along joint surfaces. To et al. (2003) proposed a method to evaluate the ultimate lateral-load capacity of shafts in rock masses with two or three sets of intersecting joints. The analysis consists of two parts. In the first part, the block theory of Goodman and Shi (1985) is used to determine if possible combinations of removable blocks exist that would represent a kinematically feasible mode of failure. In the second part, the stability of potentially removable combinations of blocks or wedges is analyzed by limit equilibrium. Both steps in the analysis require careful evaluation of the joint sets, in terms of their geometry and strength properties. Although the method is based on some idealized assumptions, such as equal joint spacing, and it has not been evaluated against field or laboratory load tests, it provides a theoretically based discontinuum analysis of stability in cases where this mode of failure requires evaluation.

Discussion of Analytical Models for Laterally Loaded Sockets

Each of the analytical methods described above has advantages and disadvantages for use in the design of rock-socketed shafts for highway bridge structures. The greatest need for further development of all available methods is a more thorough database of load test results against which existing theory can be evaluated, modified, and calibrated.

The simple closed-form expressions given by Carter and Kulhawy (1992) represent a convenient, first-order approximation of displacements and rotations of rock-socketed shafts. Advantages include the following:

- Predicts lateral displacements under working load conditions,
- Requires a single material parameter (rock mass modulus),
- Provides reasonable agreement with theoretically rigorous finite-element analysis, and
- Is the easiest method to apply by practicing design engineers.

Limitations include:

- Does not predict the complete lateral load-displacement curve,
- Elastic solution does not provide shear and moment distribution for structural design,
- Does not account for more than one rock mass layer,
- Does not account directly for nonlinear *M–EI* behavior of reinforced-concrete shaft, and
- Does not account for interaction between axial and lateral loading and its effects on structural behavior of the shaft.

The method can be best used for preliminary design; for example, establishing the initial trial depth and diameter of rock-socketed shafts under lateral and moment loading. For some situations, no further analysis may be necessary. Final design should be verified by field load testing.

The method of Zhang et al. (2000) provides a more rigorous continuum-based analysis than that of Carter and Kulhawy. The tradeoff is that more material parameters are required as input. Variation of rock mass modulus with depth is required. To fully utilize the nonlinear capabilities, the Hoek–Brown yield criterion parameters are required, and these are based on establishing the RMR and/or GSI. The method is best applied when a more refined analysis is required and the agency is willing to invest in proper determination of the required material properties. Advantages include:

- Predicts the full, nonlinear, lateral load-deformation response;
- Accounts for partial yield in either the rock mass or the overlying soil (more realistic);

- Is based on well-established rock mass and soil properties;
- Is verified against rigorous theory, for elastic range; and
- Provides shear and moment distribution for structural design.

Limitations include:

- Requires numerical (computer) solution, not currently available commercially;
- Requires a larger number of rock mass material parameters;
- Currently is limited to two layers (one soil and one rock mass layer, or two rock mass layers); and
- Nonlinear *M–EI* behavior of reinforced-concrete shaft is not accounted for explicitly; requires iterative analyses with modified values of *EI*.

The most rigorous analytical methods based on a continuum approach are FEM. When implemented by competent users, FEM analysis can account for the shaft, soil, and rock mass behaviors more rigorously than the approximate methods described herein, but FEM analyses are not suitable for routine design of foundations in most cases. First, the results are only as reliable as the input parameters. In most cases the material properties of the rock mass are not known with sufficient reliability to warrant the more sophisticated analysis. Second, the design engineer should have the appropriate level of knowledge of the mathematical techniques incorporated into the FEM analyses. Finally, the time, effort, and expense required for conducting FEM analyses are often not warranted. For very large or critical bridge structures, sophisticated FEMs may be warranted and the agency might benefit from the investment required in computer codes, personnel training, and field and laboratory testing needed to take advantage of such techniques.

Subgrade reaction methods, as implemented through the *p-y* curve method of analysis, offer some practical advantages for design. These include:

- Predicts the full, nonlinear lateral load-deformation response;
- Can incorporate multiple layers of soil and/or rock;
- Accounts for nonlinear *M–EI* behavior of reinforced-concrete shaft;
- Provides structural analysis (shear, moment, rotation, and displacement) of the drilled shaft;
- Accounts for the effects of axial compression load on the structural behavior of the shaft; and
- Can be implemented easily on a desktop computer with available software.

The principal limitations are:

- Lack of a strong theoretical basis for *p-y* curves and
- Requires back analysis of instrumented load tests to verify and validate *p-y* curves; such verification is currently lacking or limited to a few cases.

Considering that the *p-y* method is currently being used extensively by most state DOTs, effort should be made to address its present limitation by research aimed at better establishing methods to specify appropriate *p-y* curves in rock. Full-scale field load testing with instrumentation is the only known method to verify *p-y* curves. Research conducted for this purpose would provide an opportunity to evaluate and calibrate other proposed analytical methods; for example, those of Carter and Kulhawy (1992) and Zhang et al. (2000) and for development of new models. Recommendations for research are discussed further in chapter five. The research programs sponsored by the North Carolina and Ohio DOTs illustrate the type of approach that is useful for advancing all of the available methods of analysis. In addition to providing improved criteria for *p-y* curve modeling, the load test results reported by Gabr et al. (2002) and Liang and Yang (2006) can be used to evaluate each other's models and the SW and continuum models described in this chapter.

In summary, a range of analytical tools are available to foundation designers to consider rock sockets under lateral and moment loading. These include simple, closed-form equations requiring a small number of material properties (Carter and Kulhawy 1992). A more rigorous model that predicts the complete nonlinear response but requires more material properties is also available (Zhang et al. 2000). Highly sophisticated numerical models requiring extensive material properties and appropriate expertise (FEM analysis) exist and may be appropriate for larger projects. The *p-y* method of analysis is attractive to designers, as evidenced by its wide use; however, considerable judgment is required in selection of *p-y* curve parameters. All of the currently available methods suffer from a lack of field data for verification and are best applied in conjunction with local and agency experience, thorough knowledge of the geologic environment, and field load testing.

STRUCTURAL ISSUES

Twenty of the questionnaire responses indicated that structural design of drilled shaft foundations is carried out by engineers in the Bridge Design or Structures Division of their state DOTs. Three states indicated that structural design is a joint effort between the Geotechnical and Structural/Bridge Divisions. One DOT indicated that structural design is done by the Geotechnical Branch. All of the states responding to the structural design portion of the questionnaire stated that the AASHTO *LRFD Bridge Design Specifications* are followed for structural design of drilled shafts. Three states also cited the ACI *Building Code Requirements for Structural Concrete*.

Barker et al. (1991) discussed the structural design of reinforced-concrete shafts and have several design examples illustrating the basic concepts. O'Neill and Reese (1999) also covered the general aspects of reinforced-concrete design for drilled shafts, for axial compression loading and flexure, citing as primary references the 1994 AASHTO *LRFD Bridge Design Specifications* (1st edition) and the 1995 ACI *Building Code Requirements for Structural Concrete* (ACI 318-94). Both the AASHTO and ACI codes have since been revised (AASHTO in 2004 and ACI in 2002); however, there are no major differences that would change the structural design of drilled shafts. According to the survey, all of the states are designing in accordance with the AASHTO *LRFD Bridge Design Specifications*. At the time of this writing, Section 10 (Foundations) of the draft 2006 Interim AASHTO *LRFD Bridge Design Specifications* was available for reference. However, the other sections of the 2006 Interim specifications were not available and so comments pertaining to Section 5 are referenced to the 2004 edition. Only issues of structural design pertaining specifically to rock-socketed drilled shafts are addressed here.

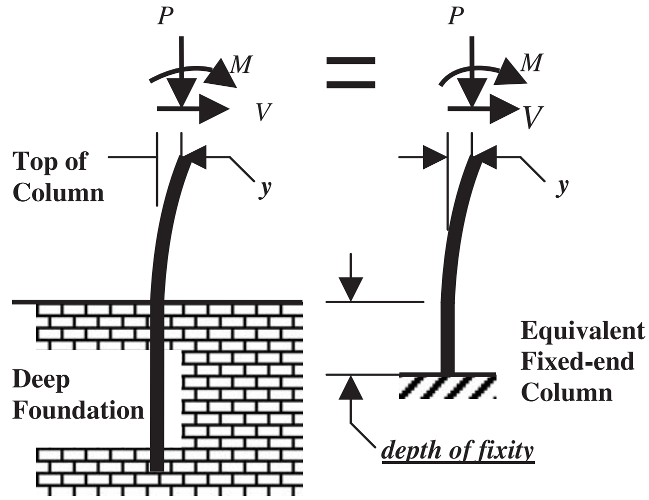


FIGURE 47 Depth of fixity for equivalent fixed-end column.

General Issues

Section 10.8.3.9 ("Shaft Structural Resistance") of the 2006 Interim AASHTO *LRFD Bridge Design Specifications* states that

> The structural design of drilled shafts shall be in accordance with the provisions of Section 5 for the design of reinforced concrete.

This language makes it clear that drilled shaft structural design is subject to the same provisions as other reinforced-concrete members. The designer must then determine whether the shaft is a compression member or a member subjected to compression and flexure (beam column). Article 10.8.3.9.3 states the following:

> Where the potential for lateral loading is insignificant, drilled shafts may be reinforced for axial loads only. Those portions of drilled shafts that are not supported laterally shall be designed as reinforced-concrete columns in accordance with Article 5.7.4. Reinforcing steel shall extend a minimum of 10 ft below the plane where the soil provides fixity.

The commentary accompanying Article 10.8.3.9.3 states further that:

> A shaft may be considered laterally supported: below the zone of liquefaction or seismic loads, in rock, or 5.0 ft below the ground surface or the lowest anticipated scour elevation. . . . . Laterally supported does not mean fixed. Fixity would occur somewhere below this location and depends on the stiffness of the supporting soil.

The language in this provision could be improved by providing a definition of "fixity." Fixity is defined by Davisson (1970) for piles under lateral loading as the depth below groundline corresponding to the fixed base of an equivalent free-standing column; that is, a column for which the top deflection and rotation would be the same as that of a column supported by the embedded deep foundation (Figure 47). Approximate equations are given by Davisson for establishing depth of fixity based on the depth of the foundation and a relative stiffness factor that depends on the flexural rigidity of the pile and the subgrade modulus of the soil or rock. Interviews with state DOT engineers indicated that different criteria for establishing depth of fixity are being applied. One state DOT defines fixity as the depth at which LPILE analysis shows the maximum moment, whereas another defines fixity as the depth at which LPILE shows zero lateral deflection. In Section 12 of *Bridge Design Aids* (1990), the Massachusetts Highway Department) describes a rigorous approach involving use of the program LPILE (or other *p-y* analysis) to establish a depth of fixity as defined in Figure 47. For the given soil/rock profile, approximate service loads are applied to the "Top of Column" (Figure 47). Shear and moment are applied as separate load cases and the resulting lateral deflections and rotations at the top of the column are designated as follows:

δ_V = deflection due to shear (V)
δ_M = deflection due to moment (M)
θ_V = rotation due to shear (V)
θ_M = rotation due to moment (M).

Equivalent column lengths are then calculated using the following analytical expressions for each loading case. The four resulting values of L should be approximately equal and the average value can be taken as a reasonable approximation of the equivalent fixed-end column length. Depth of fixity corresponds to the portion of the fixed-end column below groundline.

$$L_{\delta V} = \left[\frac{3\delta_V(EI)}{V}\right]^{1/3} \tag{136}$$

$$L_{\delta M} = \left[\frac{2\delta_M(EI)}{M}\right]^{1/2} \tag{137}$$

$$L_{\theta V} = \left[\frac{2\theta_V (EI)}{V}\right]^{1/2} \tag{138}$$

$$L_{\theta M} = \frac{\theta_M (EI)}{M} \tag{139}$$

The principal use of depth of fixity is to establish the elevation of equivalent fixed-end columns supporting the superstructure, thus enabling structural designers to uncouple the foundations from the superstructure for the purpose of structural analysis and design of the bridge or other structure. Structural modeling of the superstructure with equivalent fixed-end columns is also used to establish the column loads. These column loads are then used to analyze the drilled shaft foundations by applying them to the top of the actual column, which is continuous with the foundation (left side of Figure 47) using *p-y* analysis. As described at the beginning of this chapter, these analyses may be done by either the GD or Bridge offices, but the soil and rock parameters are provided by GD. The *p-y* analysis gives the maximum moment and shear that are used in the reinforced-concrete design. Use of software such as LPILE, COM624, or other programs is thus seen to be an integral tool in both the geotechnical and structural design of drilled shafts for bridges or other transportation structures. As noted previously, AASHTO specifications define the strength limit state for lateral loading only in terms of foundation structural resistance. Lateral deflections as predicted by *p-y* analyses are used as a design tool to satisfy service limit state criteria.

The concept of fixity also has implications for reinforcing steel requirements of drilled shafts. According to Article 10.8.3.9.3, as cited earlier, if a drilled shaft designed for axial compression extends through soil for a distance of at least 3 m (10 ft) beyond fixity before entering into rock, the rock-socketed portion of the shaft does not require reinforcement. This provision would also limit the need for compression steel in rock sockets to a maximum depth of 3 m below fixity. Exceptions to this are shafts in Seismic Zones 3 and 4, for which Article 5.13.4.6.3d states that "for cast-in-place piles, longitudinal steel shall be provided for the full length of the pile."

Some state DOTs use permanent steel casing in the top portion of drilled shafts or, in many cases, down to the top of rock. Permanent casing is not mentioned in the 2006 Interim specifications, but the 2004 specifications included the following statement: "Where permanent steel casing is used and the shell is smooth pipe greater than 0.12 in. thick, it may be considered to be load-carrying. Allowance should be made for corrosion."

A few states indicated that questions arise in connection with relatively short sockets in very hard rock. The questions pertain to moment transfer, development length of steel reinforcing, and apparently high shear loads resulting from high moment loading.

Moment Transfer

Rock sockets subjected to high lateral and/or moment loading require a minimum depth of embedment to transfer moment to the rock mass and to satisfy minimum development length requirements for reinforcing steel. The mechanism of moment transfer from a column to the rock is through the lateral resistance developed between the concrete shaft and the rock. The resistance depends on many of the factors identified previously, primarily strength and stiffness of the rock mass and flexural rigidity of the shaft. When the strength and modulus of the rock mass are greater than that of the concrete shaft, the question may arise, why excavate such high quality rock and replace it with lower strength concrete? The only means to transfer moment into the rock mass is through a properly designed shaft with the dimensions, strength, and stiffness to transmit the design moment by the assumed mechanisms of lateral resistance. In some situations where high strength rock mass is close to the ground surface, shaft size may be governed by structural considerations rather than by geotechnical capacity.

For some relatively short, stubby shafts in hard rock, socket length could be governed by the required development length of longitudinal reinforcing bars. Article 5.11 of the AASHTO *LRFD Bridge Design Specifications* (2004) specifies basic tension development lengths for various bar sizes as a function of steel and concrete strengths. Because the bars will be stressed to their maximum values at the points where maximum moments occur, the distance between the point of maximum moment and the bottom of the socket must be at least equal to the required development length. As an example, for No. 18 bars, assuming f_y = 414 MPa (60 ksi) and f_c' = 27.6 MPa (4 ksi), basic development length is 267 cm (105 in. or 8.75 ft). Although this is not often the governing factor for socket length, it should be checked.

Shear

Some designers commented on cases where *p-y* analysis of laterally loaded rock-socketed shafts resulted in unexpectedly high values of shear and whether the results were realistic. In particular, when a rock socket in relatively strong rock is subjected to a lateral load and moment at its head, values of shear near the top of the socket may be much higher than the applied lateral load. This result would be expected given the mechanism of moment load transfer. When the lateral load has a high moment arm, such as occurs in an elevated structure, the lateral load transmitted to the top of the drilled shaft may be small or modest, but the moment may be relatively large. The principal mechanism of moment transfer from the shaft to the rock mass is through the mobilized lateral resistance. If a large moment is transferred over a relatively short depth, the lateral resistance is also concentrated over a relatively short length of the shaft and results in shear loading that may be higher in magnitude than that of the lateral load.

There is some question, however, whether high values of shear predicted by *p-y* methods of analysis for such cases exist in reality or are artifacts of the analysis. One designer suggested that the structural model of the shaft does not account properly for shear deformation, resulting in unrealistically high shear values. The topic requires further investigation.

In some cases, the magnitude of shear must be addressed in the reinforced-concrete design, primarily in the use of transverse reinforcement. According to AASHTO *LRFD Bridge Design Specifications* (2004), the minimum amount of spiral reinforcement to satisfy the requirements for compression is governed by

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y} \tag{140}$$

in which ρ_s = ratio of spiral reinforcement to total volume of concrete core, measured out-to-out of spirals; A_g = gross (nominal) cross-sectional area of concrete; and A_c = cross-sectional area of concrete inside the spiral steel. When shear occurs in addition to axial compression, the section is then checked by comparing the factored shear loading with the factored shear resistance, given by

$$V_r = \phi V_n = \phi\left(V_c + V_s\right) \tag{141}$$

in which V_r = factored shear resistance, V_n = nominal shear resistance, V_c = nominal shear resistance provided by the concrete, V_s = nominal shear resistance provided by the transverse steel, and ϕ = resistance factor = 0.90 for shear. The nominal shear strength provided by the concrete is given (in U.S. customary units) by:

$$V_c = 2\left(1 + \frac{P_u}{2{,}000A_g}\right)\sqrt{f'_c}\,A_v \tag{142}$$

or, when axial load is zero,

$$V_c = 2\sqrt{f'_c}\,A_v \tag{143}$$

where P_u = factored axial load and A_v = area of concrete in the cross section that is effective in resisting shear. For a circular section this can be taken as

$$A_v = 0.9B\left[\frac{B}{2} + \frac{B_{ls}}{\pi}\right] \tag{144}$$

in which B = shaft diameter and B_{ls} = diameter of a circle passing through the center of the longitudinal reinforcement.

The nominal shear strength provided by transverse reinforcement is given by

$$V_s = \frac{A_{vs} f_y d}{s} \tag{145}$$

where A_{vs} = area of shear reinforcement, s = longitudinal (vertical) spacing of the ties or pitch of the spiral, and d = effective shear depth. For a circular cross section this can be taken as

$$d \approx 0.9\left[\frac{B}{2} + \frac{B_{ls}}{\pi}\right] \tag{146}$$

The need for additional transverse reinforcement, beyond that required for compression, can be determined by Eq. 141. For the majority of rock-socketed shafts, the transverse reinforcement required to satisfy compression criteria (Eq. 140) combined with the shear resistance provided by the concrete (Eq. 142) will be adequate to resist the factored shear loading without the need for additional transverse reinforcement. However, in cases where high lateral load or moment are to be distributed to the ground over a relatively small distance; for example, a short stubby socket in high-strength rock, factored shear forces may be high and the shaft dimensions and reinforcement may be governed by shear. In these cases, the designer is challenged to provide a design that provides adequate shear resistance without increasing the costs excessively or adversely affecting constructability by constricting the flow of concrete.

To handle high shear loading in the reinforced-concrete shaft, the designer has several options: (1) increase the shaft diameter, thus increasing the area of shear-resisting concrete; (2) increase the shear strength of the concrete; or (3) increase the amount of transverse reinforcing, either spiral or ties, to carry the additional shear. Each option has advantages and disadvantages.

Two variables that can be adjusted to increase shear resistance are concrete 28-day compressive strength, f'_c, and shaft diameter, B. Increasing the concrete strength can be a cost-effective means of increasing shear strength. For example, increasing f_c' from 27.6 MPa to 34.5 MPa (4000 psi to 5000 psi) yields a 12% increase in shearing resistance. Increasing the diameter of a rock socket can add considerably to the cost, depending on rock type, drillability, socket depth, etc. Rock of higher strength, which is likely to coincide with the case when shear is critical, can be some of the most expensive rock to drill. However, increasing the diameter can provide other benefits that may offset additional costs, such as reducing the congestion of reinforcement steel (improved constructability), increasing axial and bending capacity, and further limiting displacements.

Shear strength of the shaft can also be increased by providing additional transverse reinforcement in the form of either spiral or ties. From Eq. 145, this can be achieved by increasing the size of transverse reinforcement or by decreasing the pitch(s). Constructability can be affected when bar spacings are too small to allow adequate flow of concrete.

One aspect of reinforced-concrete behavior in shear that is not taken into account in any building code is confining stress. Shear capacity of concrete is increased at higher confining stress and deep foundations are subjected to significant confinement, especially when they are embedded in rock. This is a topic that warrants research but has yet to be investigated in a meaningful way that can be applied to foundation design.

Axial

When lateral loading is not significant, structural design of concrete shafts must account for axial compression or tension (e.g., uplift) capacity. For shafts designed for significant load transfer at the base, compression capacity of the reinforced-concrete shaft could be less than that of the rock bearing capacity. In high-strength intact rock, compressive strength of the shaft may be the limiting factor. For design of reinforced-concrete columns for axial compression, the AASHTO-factored axial resistance is given by

$$P_r = \phi 0.85\left[0.85 f'_c \left(A_g - A_{st}\right) + f_y A_{st}\right] \tag{147}$$

in which P_r = factored axial resistance, with or without flexure; ϕ = resistance factor (0.75 for columns with spiral transverse reinforcement, 0.70 for tied transverse reinforcement); f'_c = strength of concrete at 28 days; A_g = gross area of the section; A_{st} = total area of longitudinal reinforcement; and f_y = specified yield strength of reinforcement. One source of uncertainty is that the design equations given here are for unconfined reinforced-concrete columns. The effect of confinement provided by rock on the concrete strength is not easy to quantify, but increases the strength compared with zero confinement, and warrants further investigation.

SUMMARY

Lateral loading is a major design consideration for transportation structures and in many cases governs the design of rock-socketed drilled shafts. Design for lateral loading must satisfy performance criteria with respect to (1) structural resistance of the reinforced-concrete shaft for the strength limit state and (2) deflection criteria for the service limit state. Analytical methods that provide structural analysis of deep foundations while accounting for soil–structure interaction have, therefore, found wide application in the transportation field. However, the ability of analytical methods to account properly for rock mass response and rock–structure interaction has not developed to the same level as methods used for deep foundations in soil.

The survey shows that most state DOTs use the program COM624 or its commercial version LPILE for design of rock-socketed shafts. Review of the *p-y* curve criteria currently built into these programs for modeling rock mass response shows that they should be considered as "interim" and that research is needed to develop improved criteria. Some of this work is underway and research by North Carolina (Gabr et al. 2002), Ohio (Liang and Yang 2006), Florida (McVay and Niraula 2004), and Ashour et al. (2001) is described. All of these criteria are in various stages of development and are not being applied extensively.

Models based on elastic continuum theory and developed specifically for rock-socketed shafts have been published. Two methods reviewed in this chapter are the models of Carter and Kulhawy (1992) and Zhang et al. (2000). Advantages and disadvantages of each are discussed and compared with *p-y* methods of analysis. These models are most useful as first-order approximations of shaft lateral displacements for cases where the subsurface profile can be approximated as consisting of one or two homogeneous layers. For example, when a preliminary analysis is needed to develop trial designs that will satisfy service limit state deflection criteria, the method of Carter and Kulhawy can provide convenient solutions that can be executed by means of spreadsheet analysis. A disadvantage of these methods is that they do not directly provide solutions to maximum shear and moment, parameters needed for structural design, and they do not incorporate directly the nonlinear properties of the reinforced-concrete shaft.

Structural issues associated with rock-socketed shafts are reviewed. The concept of depth of fixity is shown to be a useful analytical tool providing a link between geotechnical and structural analysis of drilled shafts. A method for establishing depth of fixity is presented and its use in the design process is described. Other issues identified by the survey, including high shear in short sockets subjected to high moment loading and its implications for reinforced-concrete design, are addressed.

CHAPTER FIVE

CONSTRUCTION AND FIELD TESTING

SCOPE

Construction, inspection, post-construction integrity testing, and load testing of drilled shafts are related directly to design and performance. These activities are carried out in the field and depend on the skill and experience of contractors, technicians, inspection personnel, and engineers. In this chapter an overview is presented of construction methods for rock sockets. Methods for load testing of rock-socketed shafts are reviewed, including several innovative methods that have made load testing more accessible to state transportation agencies. Illustrative examples demonstrate how load testing can contribute to the economical design of rock sockets. Constructability issues identified by the survey questionnaire are discussed, and practices that can lead to quality construction are identified. Current practice for inspection and quality assurance methods for rock-socketed shafts are also reviewed and discussed. Finally, special geologic conditions that pose unique challenges for design and construction of rock sockets are described, and approaches for using rock sockets successfully in such environments are identified.

CONSTRUCTION OF ROCK SOCKETS

The art and science of drilled shaft construction are as important to the success of a bridge foundation project as are the analytical methods used to design the shafts. Construction of shafts in rock can be some of the most challenging and may require special expertise and equipment. Experience demonstrates that the key components of success are: (1) adequate knowledge of the subsurface conditions, for both design and construction; (2) a competent contractor with the proper equipment to do the job; and (3) a design that takes into account the constructability of rock sockets for the particular job conditions. Publications that cover drilled shaft construction methods include Greer and Gardner (1986) and O'Neill and Reese (1999). Aspects of construction that are related to rock sockets are reviewed herein.

Drilling Methods and Equipment

Most rock-socketed shafts are excavated using rotary drilling equipment. A rotary drill may be mechanically driven or use hydraulic motors. Mechanically driven rigs deliver power to a stationary rotary table that rotates a kelly bar to which excavation tools are attached. Mechanically driven rigs can be truck-mounted or attached to a crane (Figure 48). Hydraulic drilling rigs are equipped with hydraulic motors that can be moved up and down the mast and are usually truck or crawler mounted. Smaller hydraulic units can be mounted on an excavator. Hydraulic drilling rigs with significantly increased power have appeared in the North American market in recent years. Drilling in rock, especially hard rock, generally requires machines with more power than for drilling in soil. Equipment with higher torque ratings and additional power has given more contractors the capability to install rock-socketed shafts than existed previously. This is a positive development for the U.S. market in that it promotes competition and expands the base of experienced contractors for rock-socket construction.

Equipment developed in Europe and now being used by some North American contractors uses hydraulic rams configured to rotate or oscillate (rotate back and forth) a steel casing into the ground (Figure 49). Soil or rock is excavated from inside the casing using a hammergrab, a percussion tool that breaks and removes soil or rock. In most cases the rotator or oscillator is bolted to a crane for stability under the large torque that must be developed. The crane can also provide hydraulic power to operate the rams that turn the casing. Rotators have the capability to cut through high-strength rock in the range of 100–150 MPa (15–22 ksi) depending on the degree of fracturing (J. Roe, Malcolm Drilling, personal communication, Oct. 3, 2005). A 3-m-diameter oscillator such as the one shown in Figure 49 is generally limited to cutting through weaker rock with strength less than 100 MPa. The lead casing on the oscillator must have teeth set in opposite directions to cut back and forth. Both methods are efficient in penetrating large cobbles and boulders, a situation common to glacial till deposits and cemented sands and gravels.

Rock Cutting Tools

Selecting the proper cutting tool depends on many variables, including rock mass properties (strength, hardness, and structure), type of drilling machine, socket depth and diameter, condition and cost of the tools, operator skill, previous experience in similar conditions, and judgment. There are no absolute rules and different contractors may take a completely different approach when faced with similar conditions. New tools and innovations are constantly being introduced. Following is a summary of some of the most common cutting tools used for rock-socket construction.

FIGURE 48 Intact rock core removed using crane-mounted rotary drill and core barrel.

When relatively stiff soil or weak rock cannot be penetrated efficiently with typical soil drilling tools (e.g., open helix augers), most contractors will attempt to use a rock auger. Rock augers are manufactured from thicker metal plate than soil augers and have cutting teeth. The teeth may be of the drag bit type, which are effective in cutting rock but wear rapidly and must be replaced frequently. As a rule of thumb, these types of teeth are limited to cutting rock of compressive strength up to approximately 48 MPa (7,000 psi), at which point they dull quickly. Conical-shaped teeth made of tungsten carbide or other alloys depend on crushing the rock and are more durable than drag bits, but require considerable downward force (crowd) to be effective. Figure 50 shows a rock auger with both types of teeth, to exploit both mechanisms

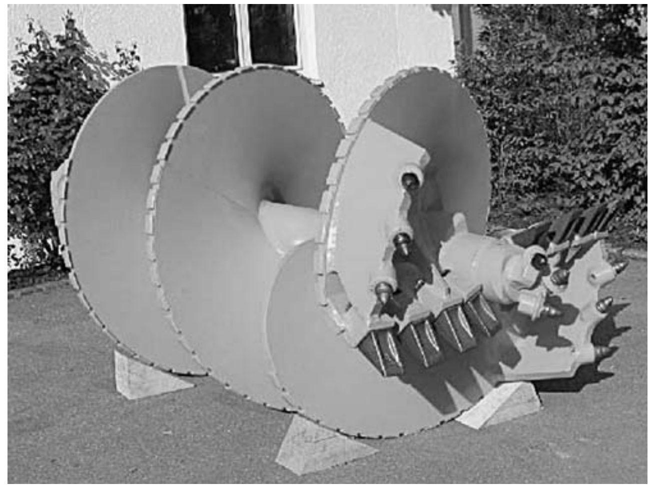


FIGURE 50 Rock auger with drag bit and bullet-shaped cutting teeth.

of cutting and crushing. Rock augers may be stepped or tapered so that the initial penetration into rock requires less torque and crowd, or the socket may be drilled first by a smaller diameter tool such as the one shown in Figure 51, followed by a larger diameter auger. This releases some of the confinement and causes less wear and tear on the drilling tools. Replacement or reconditioning of rock auger teeth can be a major contractor cost, especially in highly abrasive rock.

Self-rotating cutter bits combine a highly efficient cutting mechanism with the durability of some conical bits. A rock auger with self-rotating cutters, for excavating the face of the socket, and conical bits directed outward is shown in Figure 52. A contractor using this auger reported penetration rates two to three times higher than with conventional rock augers and in very hard (100 MPa or 15,000 psi) rock.

At some combination of rock strength and socket diameter rock augers are no longer cost-effective. One contractor

FIGURE 49 Casing oscillator and hammergrab tool.

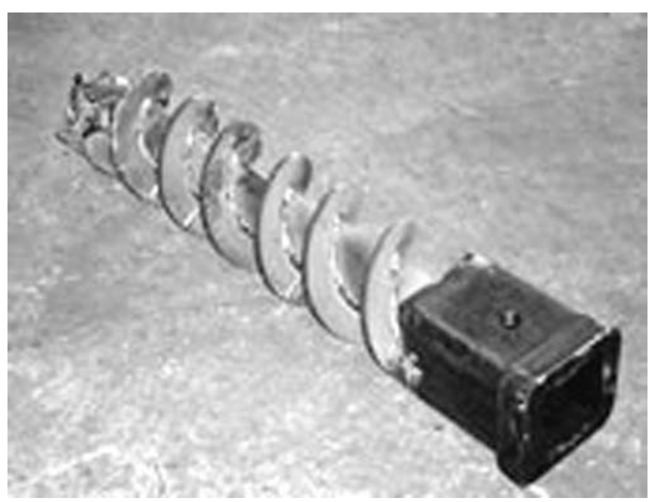


FIGURE 51 Small diameter rock auger for creating a pilot hole.

FIGURE 52 Rock auger with conical teeth and rotating cutters (*Courtesy*: V. Jue, Champion Equipment, Inc.).

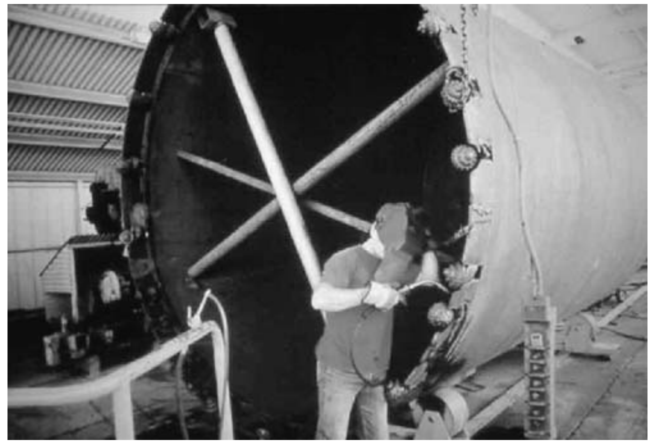


FIGURE 54 Welding roller bits on a 4-m-diameter double-walled core barrel.

interviewed for this study stated that, for socket diameters up to approximately 1.8 m and rock strength up to 70 MPa (10,000 psi), initial cost estimates are based on the assumption that rock augers will be used. If the combination of socket diameter and rock strength exceeds those values, the job is bid on the assumption that rock will be cored. Of course, these rule-of-thumb criteria are subject to change on the basis of rock mass characteristics, experience, etc., and will vary between contractors. Use of a single parameter, such as uniaxial compressive strength of rock, does not capture all of the variables that determine penetration rates for a given set of conditions.

Coring is a widely used method when rock augers are no longer feasible. The basic concept is that coring reduces the volume of rock that is actually cut by the teeth. A simple configuration consists of a single cylindrical barrel with cutting teeth at the bottom edge (Figure 53). The teeth cut a clearance on the inside and outside of the barrel that is sufficient for removing cuttings and extraction of the core barrel. The core may break off at a discontinuity or it may require use of a rock chisel, a metal tool that is wedged between the barrel and the rock to fracture the core. The core will usually jam into the barrel and can be lifted out of the hole and then removed by hammering the suspended barrel (see Figure 48). If the rock is highly fractured, the core barrel may be removed, followed by excavation of the fractured rock from the hole. For deep sockets or for harder rock, double wall core barrels may be used. The outer barrel is set with teeth, typically roller bits (Figure 54), while the core is forced into the inner barrel. Compressed air is circulated between the barrels to remove cuttings.

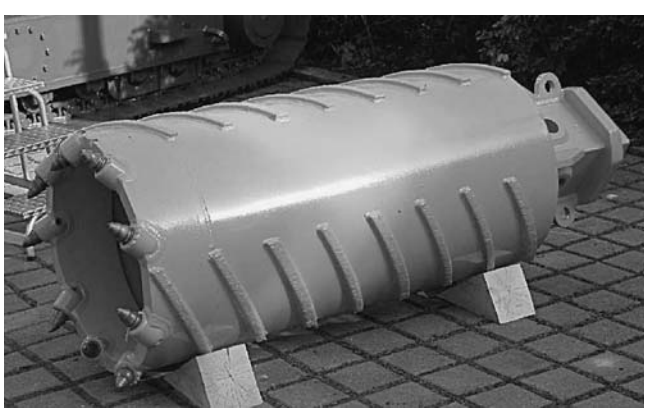


FIGURE 53 Typical single wall core barrel.

For very high strength rock ($q_u \geq 100$ MPa) there are few tools that will excavate efficiently. In these rocks, however, even a small penetration can provide high axial, and in some cases lateral, resistance. A shot barrel, in which hard steel shot is fed into the annular space between the double walls of the core barrel, may work in such conditions. Grinding action of the shot excavates the rock and water is circulated for cooling the shot.

Excavation rates with core barrels are typically slow. Although coring may be cost-effective because of the foundation performance benefits achieved, careful attention should be given to avoiding overly conservative designs that significantly increase the cost of drilled shafts made by unnecessary coring into rock.

Hard rock can also be excavated using downhole hammer bits. The tool shown in Figure 55 has an array of button-bit hammers (called a cluster drill) operated independently by compressed air. Air pressure also lifts the cuttings which are collected in a calyx basket. On the tool shown in Figure 55, some of the bits can be rotated outward to create a larger diameter socket (under reaming) than the casing, and then retracted to remove the bit. This allows a casing to be installed directly behind the bit during drilling. Downhole hammers and cluster drills are generally expensive and require large air compressors to operate. Most contractors will rent this equipment when needed, which is only cost-effective in very hard rock.

FIGURE 55 Downhole hammer tool for drilling in hard rock.

A technique used for drilling large rock sockets at the Richmond–San Rafael Bridge (Byles 2004) is reverse circulation drilling with a "pile top" rig. The unit consists of two main components. A top unit (Figure 56) is fixed to the top of a steel casing. The "bottom hole assembly" (Figure 57) is a drill bit lowered to the bottom of the hole through a casing, submerged in water or other drilling fluid. The bit is operated hydraulically through lines extending from the top unit (sometimes called a rodless drill), which is fixed to the top of the casing. Alternatively, a drill rod may be used to transmit torque from the top unit to the bit. The bit has a central hollow orifice connected to a flexible line extending back up to the top unit. During drilling, a vacuum pump or air lift is used to draw the drilling fluid with the cuttings upward to a cleaning plant, from where it is circulated back into the hole. The unit shown in Figure 56 was used to drill 3.35-m-diameter rock sockets in Franciscan Formation sandstone and serpentinite. Some manufacturers are now producing reverse circulation units that can be installed on a conventional rotary hydraulic drilling rig to provide similar capability, at a smaller diameter. It is likely that these units will become more common in North America for rock-socket drilling (D. Poland, Anderson Drilling, personal communication, Aug. 2, 2005). Reverse circulation drilling can also be carried out with any type of rotary drill rig equipped with a hollow Kelly bar (drill stem) that allows circulation of the drilling fluid from the cutting surface up through the bar.

FIGURE 56 Toredo T40-4 pile top unit being placed over casing for reverse circulation drilling, Richmond–San Rafael Bridge (California) (Byles 2004).

FIELD LOAD TESTING

The most direct method to determine the performance of full-scale rock-socketed drilled shafts is through field load testing. Clearly there have been advances in engineers' ability to predict rock-socket behavior. However, there will always be sources of uncertainty in the applicability of analysis methods, in the rock mass properties used in the analysis, and with respect to the unknown effects of construction. Load testing provides direct measurement of load displacement response for the particular conditions of the test foundation, and can also provide data against which analytical models can be evaluated and calibrated.

Objectives

Field load testing may be conducted with different objectives and this should determine the scope of testing, type of tests, and instrumentation. A partial listing of valid reasons for transportation agencies to undertake load testing of rock-socketed shafts includes:

- Confirm design assumptions,
- Evaluate rock resistance properties,
- Evaluate construction methods,
- Reduce foundation costs, and
- Research aimed at evaluating or improving design methods.

More than one of these objectives can sometimes be achieved. For example, load tests conducted primarily for confirmation of design assumptions (proof test) for a particular project can be useful to researchers by contributing additional data for evaluating empirical correlations proposed

FIGURE 57 Shrouded bottom hole assembly lifted for placement through the top unit (Byles 2004).

for design. Load tests carried to ultimate capacity of the shaft are especially valuable not only to the agency conducting the test or for the specific bridge project, but to the entire deep foundation engineering community.

The costs of conducting field load tests should be offset by its benefits. The most obvious costs include the dollar amount of contracts for conducting testing. Other costs that are not always as obvious include construction delays, delays in design schedule, and DOT person hours involved in the testing. Direct cost benefits may be possible if the testing leads to more economical designs. This requires testing prior to or during the design phase. Numerous case histories in the literature show that load testing almost always leads to savings. Lower factors of safety and higher resistance factors are allowed by AASHTO for deep foundation design when a load test has been conducted.

Other benefits may not be so obvious or may occur over time. Construction of the test shaft provides the DOT and all subsequent bidders with valuable information on constructability that can result in more competitive bids. Refinement in design methods resulting from information gained by load testing has economic benefits on future projects.

Load test results provide the most benefit when they are accompanied by high-quality subsurface characterization. Knowledge of site stratigraphy, soil and rock mass properties, site variability, and groundwater conditions are essential for correct interpretation of load test results. The ability to apply load test results to other locations is enhanced when subsurface conditions can be compared on the basis of reliable data.

Construction factors and their potential effects on shaft behavior should be considered when using load test results as the basis for design of productions shafts. Items such as construction method (casing, slurry, dry), type of drilling fluid, cleanout techniques, and others may have influenced the behavior of the test shaft. If possible, the construction methods anticipated for production shafts should be used to construct test shafts.

Axial Load Testing

Conventional Axial Load Testing

Until the early 1990s the most common procedure for conducting a static axial compression load test on a deep foundation followed the ASTM Standard Method D1143, referred to herein as a conventional axial load test. Several load application methods are possible, but the most common involves using either (1) a hydraulic jack acting against a reaction beam that is anchored against uplift by piles or (2) a loading platform over the pile top on which dead load is placed. Six states indicated that they have conducted conventional axial load tests on rock-socketed shafts. Conduct and interpretation of axial compression and uplift load tests specifically for drilled shafts is discussed in detail by Hirany and Kulhawy (1988).

Axial load tests may be conducted for the purpose of confirming the design load for a specific project, in which case it is typical to load the shaft to twice the anticipated design load to prove the shaft can support the load with an acceptable settlement (a proof load test). This type of test is normally conducted under the construction contract and does not yield a measured ultimate capacity, unless the shaft fails, in which case the design must be adjusted. Proof tested shafts normally are not instrumented except to measure load and displacement at the head of the shaft. When the objective of testing is to gain information on behavior of the shaft in terms of load transfer, the shaft should be instrumented to determine the distribution of axial load as a function of depth and as a function of axial deformation.

Common types of instrumentation for measuring axial load and deformation at specific points along the length of the shaft include sister bars and telltales. A sister bar is a section of reinforcing steel, typically 1.2 m in length, with a strain gage attached in the center. Either vibrating wire or electrical resistance-type gages can be used. The sister bar is tied to steel of the reinforcing cage and its lead wires are routed to the surface, where they are monitored by a computer-controlled data acquisition unit. The gage signals are converted to strain, which is assumed to be equal to the strain in the concrete and can be used to estimate load using the appropriate elastic modulus and section properties of the shaft. A telltale is a metal rod installed within a hollow tube embedded in the shaft. The bottom end of the rod is fixed at a predetermined depth in the shaft and is the only point on the rod in physical contact with the shaft. By measuring vertical deformation of the upper end of the telltale during loading, deformation of the shaft is determined for the depth at which the telltale is fixed. By measuring the relative displacement between two successive rods and distance between their bottom ends, the average strain in the shaft between the two telltales can be determined. Further information on these and other types of instrumentation is given by Hirany and Kulhawy (1988) and O'Neill and Reese (1999).

The following case illustrates effective use of conventional axial load test on rock sockets. Zhan and Yin (2000) describe axial load tests on two shafts for the purpose of confirming design allowable side and base resistance values in moderately weathered volcanic rock for a Hong Kong transit project. The proposed design end bearing stress (7.5 MPa) exceeded the value allowed by the Hong Kong Building Code (5 MPa). One of the objectives of load testing was, therefore, to demonstrate that a higher base resistance could be used. The project involved 1,000 drilled shafts; therefore, proving the higher proposed values offered considerable potential cost savings.

Figure 58 shows the load test arrangement, consisting of a loading platform for placement of dead load. Figure 59 shows details of one of the instrumented shafts. Strain gages were provided at 17 different levels, including 4 levels of gages in the rock socket. Two telltales were installed, one at the base of the socket and one at the top of the socket. Shafts were excavated through overburden soils using temporary casing to the top of rock. When weathered rock was encountered, a 1.35-m-diameter reverse circulation drill (RCD) was used to advance to the bearing rock, followed by a 1.05-m-diameter RCD to form the rock socket. For the shaft shown in Figure 59, the socket was 2 m in length. A permanent, bitumen-coated casing (to reduce side resistance in the overburden materials) was placed to the top of the socket. The bottom was cleaned by airlift and concrete placed by tremie (wet pour).

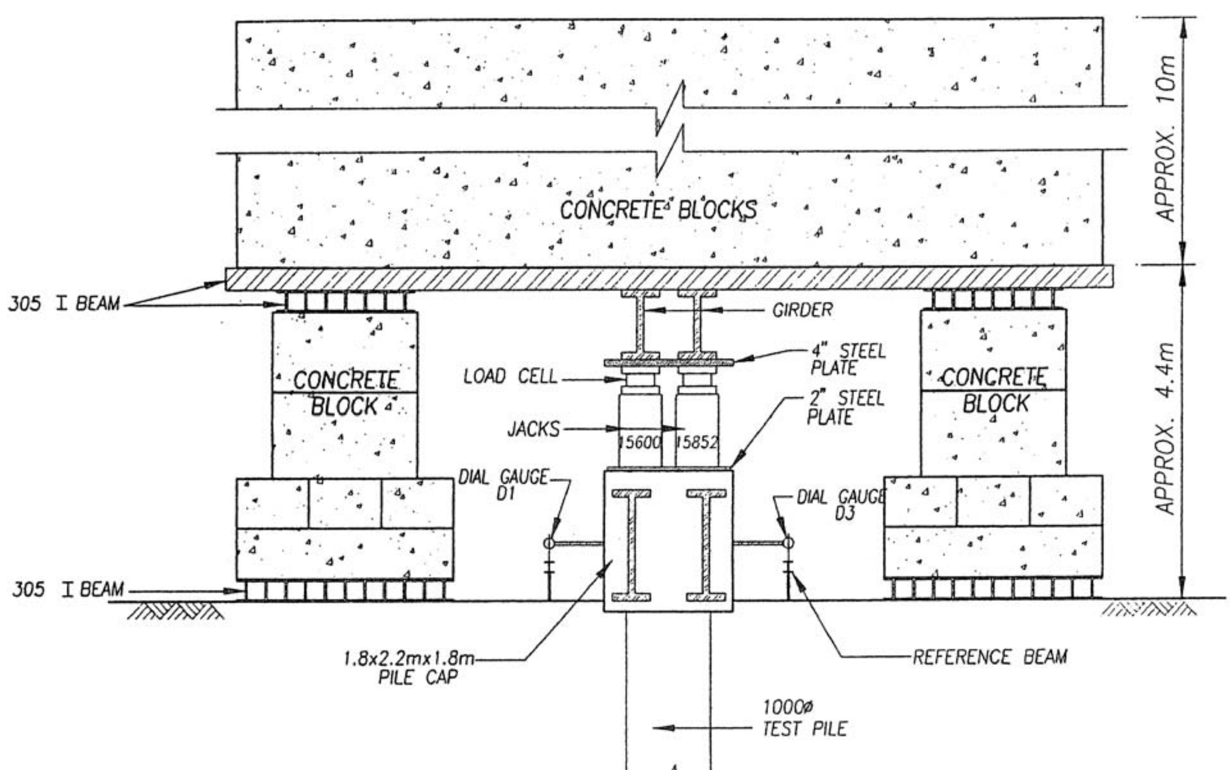


FIGURE 58 Axial load testing setup (Zhan and Yin 2000).

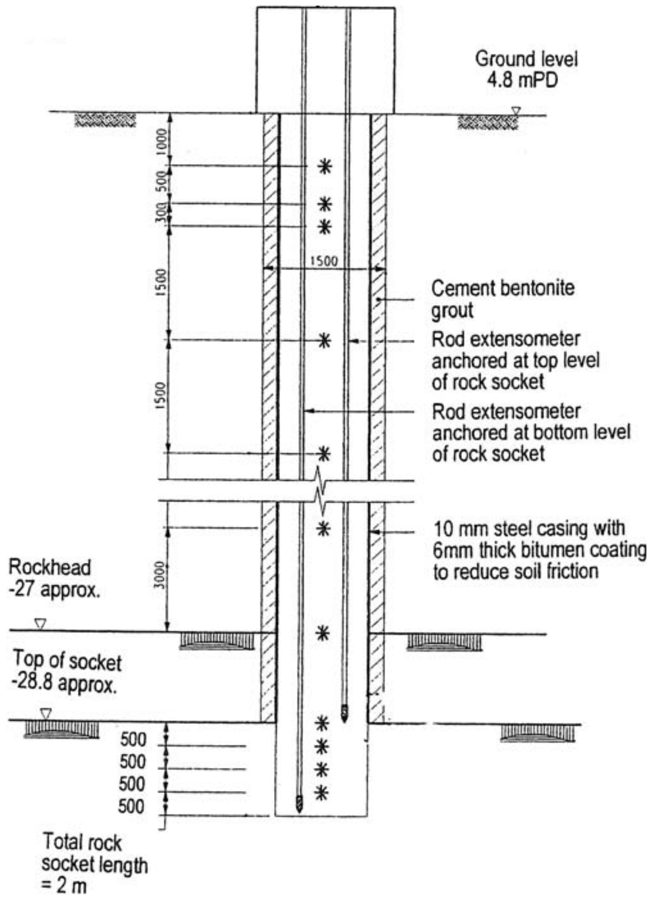


FIGURE 59 Details of instrumented rock-socketed shaft (Zhan and Yin 2000).

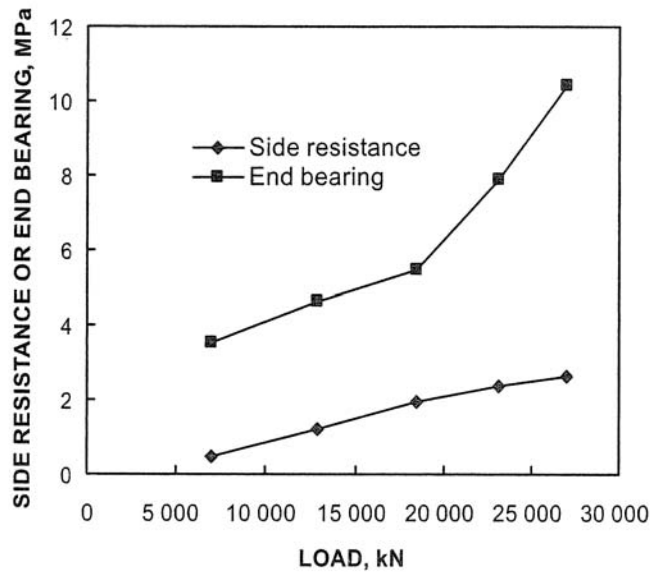


FIGURE 60 Unit side and base resistance versus axial load (Zhan and Yin 2000).

Figure 60 shows the results in terms of mobilized unit side and base resistances versus load applied at the head of the shaft. Unit side resistance reached a value of 2.63 MPa, well exceeding the proposed design allowable value of 0.75 MPa. Zhan and Yin noted that this value agrees well with Eq. 30 in chapter three. Load transfer to the base was mobilized immediately upon loading, indicating excellent base conditions, and reached a value exceeding 10 MPa. In the other shaft (not shown) a unit base resistance of 20.8 MPa was reached with no sign of approaching failure.

The case presented by Zhan and Yin demonstrates how a set of well-instrumented conventional axial load tests can be used to (1) achieve cost savings on a project with a large number of shafts, (2) confirm design allowable values of socket resistance, (3) demonstrate suitability of the construction method, and (4) provide data against which design methods can be evaluated.

Conventional axial load testing has largely been replaced by methods that are easier to set up and conduct, require less equipment and space, are safer, less time consuming, and usually less expensive, especially in rock. These methods include the O-cell, Statnamic (STN), and dynamic impact load tests. NCHRP Project 21-08, entitled "Innovative Load Testing Systems" was undertaken to evaluate these and other methods for deep foundations and to recommend interim procedures for their use and interpretation. A draft final report by Paikowsky et al. (2004b) describes these methods in detail. The role of each of these tests for rock-socketed shafts is described here.

Osterberg Load Cell

The O-cell is a hydraulically operated jacking device that can be embedded in a drilled shaft by attachment to the reinforcing cage (Figure 61). After concrete placement and curing, a load test is conducted by expanding the cell against the portions of the shaft above and below it (Osterberg 1995). The load is applied through hydraulic piston-type jacks acting against the top and bottom cylindrical plates of the cell. The

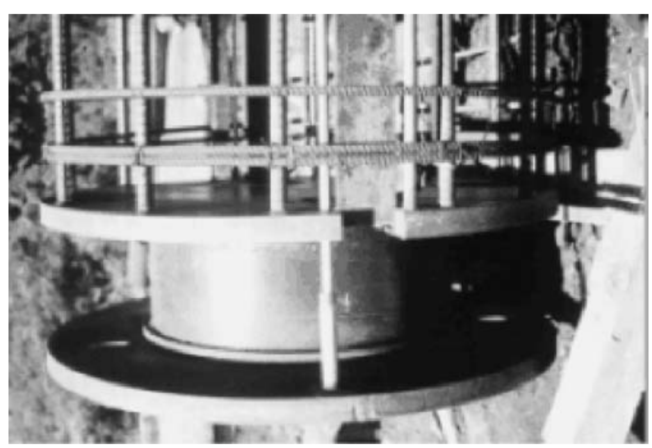


FIGURE 61 O-cell at bottom of reinforcing cage ready for placement in a drilled shaft. (O'Neill and Reese 1999).

maximum test load is limited to the ultimate capacity of either the section of shaft below the cell, the section above the cell, or the capacity of the cell.

Pressure transducers are used to monitor hydraulic jack pressures and converted to load. Linear vibrating wire displacement transducers (LVWDTs) between the two plates measure total expansion of the cell and telltales are installed to measure vertical movements at the top and bottom of the test sections. The downward movement of the bottom plate is obtained by subtracting the upward movement of the top test section from the total extension of the O-cell as determined by the LVWDTs. Telltale deformations are monitored with digital gages mounted on a reference beam. All of the instrumentation is electronic and readings are collected by a data acquisition unit.

The O-cell testing method provides some important advantages. There is no structural loading system at the ground surface. Load can be applied at or very close to the base of a socket for measurement of base resistance. In conventional top load testing, most or all of the side resistance must be mobilized before there is significant load transfer to the base. Some of the cited disadvantages are that the O-cell is sacrificial and requires prior installation, so it is not useful for testing existing foundations. Using a single O-cell, it is possible to mobilize the ultimate capacity of one portion of the shaft only, so that other sections of the shaft are not loaded to their ultimate capacity.

According to DiMillio (1998), the majority of load tests on drilled shafts are now being done with the O-cell. This is supported by results of this study, in which 17 of 32 states responding to the survey reported using the O-cell for axial load testing of rock-socketed shafts. Of these, 13 stated that ultimate side resistance was determined and 7 reported that the ultimate base resistance was determined. Five states indicated the test was used for proof load testing, in which design values of shaft resistance were verified. These responses show that the O-cell has become a widely used method for axial load testing of rock sockets.

A set of O-cell tests reported by Gunnink and Kiehne (2002) serves to illustrate the type of information that is obtained from a typical test in which a single O-cell is installed at the base of a rock socket. Figure 62 shows the test setup for three test shafts socketed into Burlington limestone. As shown, the shafts extended through soil before being socketed into limestone. All shafts were 0.46 m in diameter and socket lengths ranged from 3.45 m to 3.85 m. Depth of soil was approximately 4 m. Figure 63 shows test results for two of the shafts (Shaft Nos. 1 and 3), respectively. Each graph shows two curves, one of the O-cell load versus average measured uplift of the upper portion of the shaft, and the other of the O-cell load versus downward displacement of the base of the cell. Both figures are typical of failure of the shaft in uplift. At the maximum test load, it was not possible to main-

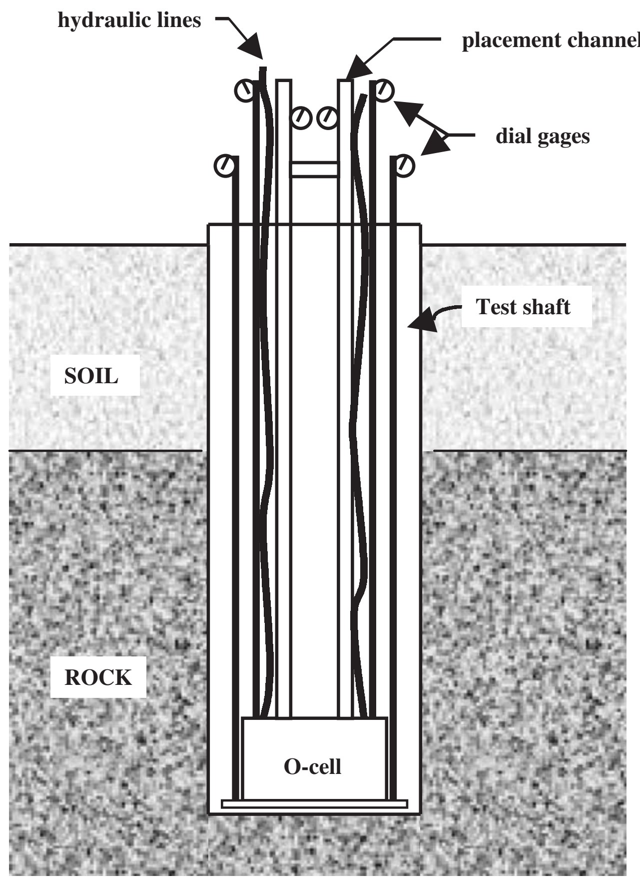


FIGURE 62 Shaft and O-cell test setup (adapted from Gunnink and Kiehne 2002).

tain or increase load without continuous upward deflection of the top of the shaft, whereas the average base displacement did not change. From these tests, it is not possible to determine ultimate base resistance values. The base load displacement curves show an interesting difference. For Shaft No. 1, the downward base movement is small (around 1 mm) up to the maximum test load, suggesting a very stiff base and good contact between the concrete and underlying rock. However, the curve for Shaft No. 3 shows downward movement approaching 10 mm upon application of the load, followed by a flattening of the curve. This behavior suggests the presence of a compressible layer between the concrete and underlying rock, possibly the result of inadequate cleanout of the hole before pouring concrete. Both shafts were poured under dry conditions and both were cleaned using the same method, reported as "rapidly spinning the auger bit after the addition of water and then lifting out the rock cuttings."

Gunnink and Kiehne (2002) reported that it is common practice to design drilled shafts founded in sound Burlington limestone for base resistance only, using a presumptive allowable unit base resistance of 1.9 MPa. Side resistance is often neglected for design. Even the lowest observed base resistance measured by the O-cell tests yielded an allowable unit base resistance of 5 MPa, assuming a factor of safety

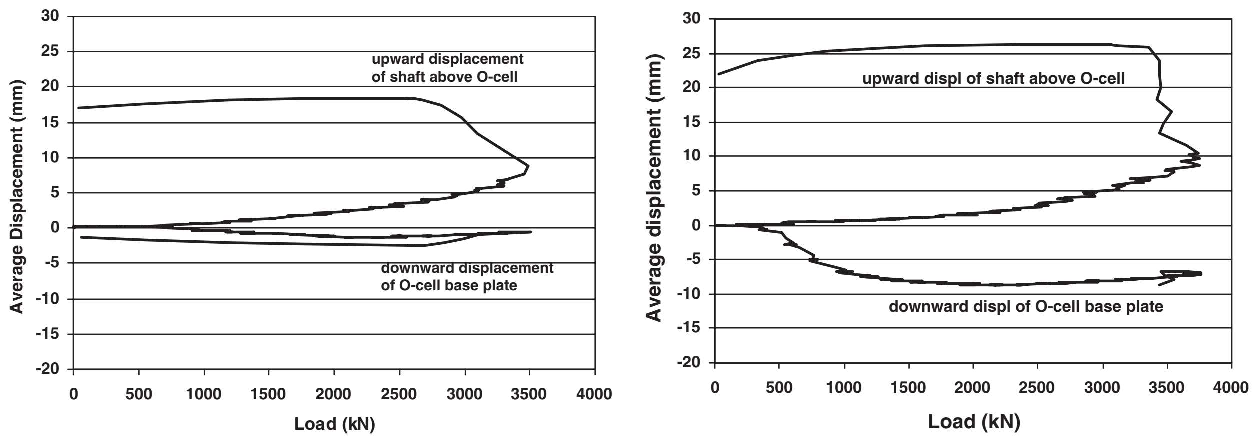


FIGURE 63 Results of single O-cell load tests: (*left*) Shaft No. 1; (*right*) Shaft No. 3 (Gunnink and Kiehne 2002).

of 3. The tests shown in Figure 63 yield ultimate unit side resistances of 2.34 and 2.28 MPa, respectively. These tests illustrate a typical outcome when field load testing is conducted; that is, measured unit side and base resistances exceed presumptive values, sometimes significantly. Load testing results make it possible to achieve more economical designs. The O-cell tests also identify construction deficiencies, such as inadequate base cleanout (Figure 63 *left*).

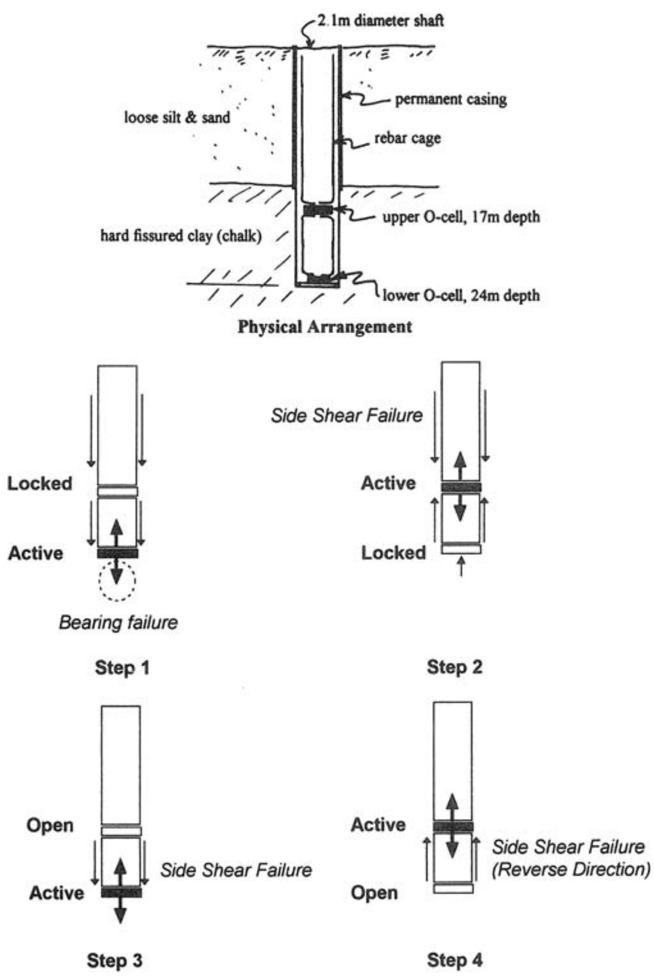


FIGURE 64 Test setup and loading sequence with two O-cells (O'Neill et al. 1997).

The tests reported by Gunnink and Kiehne also illustrate a limitation of testing with a single O-cell at the bottom of the socket. The values of ultimate unit side resistance reported by the authors are based on the assumption that all of the load was resisted by the rock socket, neglecting any contribution of the overlying soil. It is not known how significant the error is for this case, but testing with multiple O-cells makes it possible to isolate the section of shaft in rock for evaluation of average side resistance (however, multiple O-cells increase the cost of load testing). For example, if a second O-cell is located at the top of the rock socket, a test conducted with that cell can be used to determine the combined side resistance of all layers above the rock. An innovative approach based on this concept is illustrated in the testing sequence shown in Figure 64. The figure and description are from O'Neill et al. (1997) based on tests conducted by LOADTEST, Inc., for the Alabama DOT. Arrangement of the O-cells and the 4-step testing sequence depicted in the figure made it possible to measure ultimate base resistance, side resistance of the socket (in both directions), and side resistance of the cased portion of the shaft above the socket. It is noted that this arrangement made it possible to measure a total foundation resistance of 80 MN, compared with approximately 11 MN for the largest standard surface jacks. Installation of multiple O-cells makes it necessary to provide a tremie bypass line to facilitate placement of concrete below and around the upper cells.

Interpretation of O-cell tests in rock sockets is typically based on the assumption that total applied load at the ultimate condition is distributed uniformly over the shaft/rock side interface, and used to calculate an average unit side resistance by

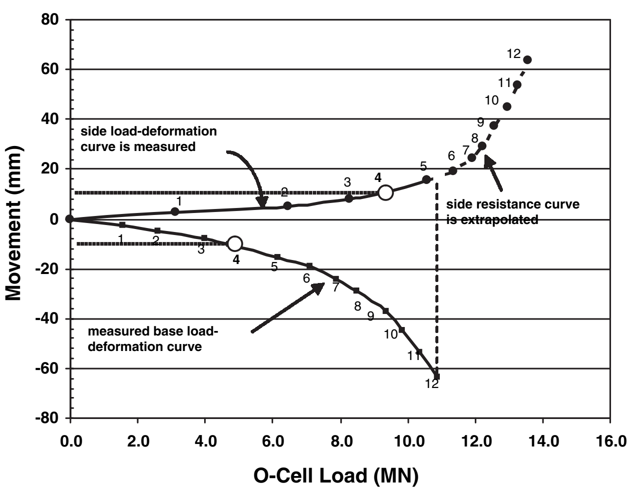
$$f_s = \frac{Q_{oc}}{\pi BD} \tag{148}$$

where

f_s = average unit side resistance (stress),
Q_{OC} = O-cell test load,
B = shaft diameter, and
D = socket length.

The degree to which this average unit side resistance is valid for design of rock sockets loaded at the head depends on the degree to which side load transfer under O-cell test conditions is similar to conditions under head loading. Detailed knowledge of site stratigraphy is needed to interpret side load transfer.

O-cell test results typically are used to construct an equivalent top-loaded settlement curve, as illustrated in Figure 65. At equivalent values of displacement both components of load are added. For example, in Figure 65*a*, the displacement for both points labeled "4" is 10 mm. The measured upward and downward loads determined for this displacement are added to obtain the equivalent top load for a downward displacement of 10 mm and plotted on a load-displacement curve as shown in Figure 65*b*. This procedure is used to obtain points on the load-displacement curve up to a displacement corresponding to the least of the two values (side or base displacement) at the maximum test load. In Figure 65*a*, this corresponds to side displacement. Total resistance corresponding to further displacements is approximated as follows. For the section of shaft loaded to higher displacement, the actual measured load can be determined for each value of displacement up to the maximum test load (in Figure 65*a* this is the base resistance curve). The resistance provided by the other section must be estimated by extrapolating its curve beyond the maximum test load. In Figure 65*a*, the side resistance curve is extrapolated. The resulting equivalent top-loaded settlement curve shown in Figure 65*b* is therefore based on direct measurements up to a certain point, and partially on extrapolated estimates beyond that point.



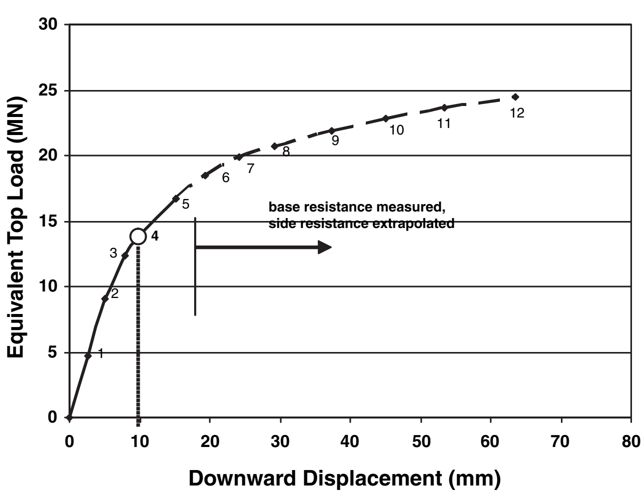


FIGURE 65 Construction of equivalent top-loaded settlement curve from O-cell test results (*a*) O-cell measured load-displacement; (*b*) equivalent top-load settlement results.

According to Paikowsky et al. (2004b), most state DOT geotechnical engineers using O-cell testing tend to accept the measurements as indicative of drilled shaft performance under conventional top-down loading. O-cell test results are applied in design by construction of an equivalent top-load settlement curve, as illustrated earlier, or by using the measured unit side and base resistances as design nominal values. However, some researchers (O'Neill et al. 1997; Paikowsky et al. 2004b) have pointed out differences between O-cell test conditions and top loading conditions that may require interpretation. The most significant difference is that compressional loading at the head of a shaft causes compression in the concrete, outward radial strain (Poisson's effect), and a load transfer distribution in which axial load in the shaft decreases with depth. Loading from an embedded O-cell also produces compression in the concrete, but a load transfer distribution in which axial load in the shaft decreases upward from a maximum at the O-cell to zero at the head of the shaft. It is possible that different load transfer distributions could result in different distributions of side resistance with depth and, depending on subsurface conditions, different total side resistance of a rock socket.

In shallow rock sockets under bottom-up (O-cell) loading conditions, a potential failure mode is by formation of a conical wedge-type failure surface ("cone breakout"). This type of failure mode would not yield results equivalent to a shaft loaded in compression from the top. A construction detail noted by Crapps and Schmertmann (2002) that could potentially influence load test results is the change in shaft diameter that might exist at the top of a rock socket. A common practice is to use temporary casing to the top of rock, followed by a change in the tooling and a decrease in the diameter of the rock socket relative to the diameter of the shaft above the socket. Top-down compression loading produces perimeter bearing stress at the diameter change as illustrated in Figure 66, whereas loading from an O-cell at the bottom of the socket would lift the shaft from the bearing surface.

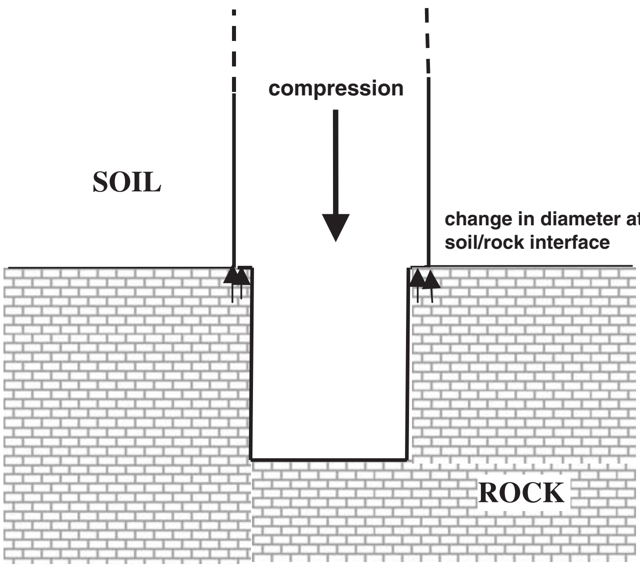


FIGURE 66 Perimeter bearing stress at diameter change under top loading.

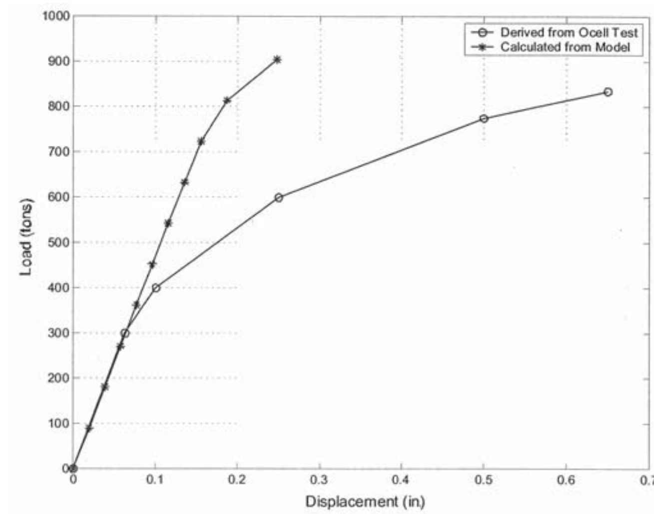


FIGURE 67 Comparison of load-displacement curves; O-cell versus FEM (Paikowsky et al. 2004b).

Paikowsky et al. (2004b) reviewed the available data that might allow direct comparisons between O-cell and conventional top-down loading tests on drilled shafts. Three sets of load tests reported in the literature and involving rock sockets were reviewed. However, in two of the cases the test sequence involved conventional top-down compression loading (Phase 1) followed by O-cell testing from the bottom up (Phase 2). Mobilization of side resistance in Phase 1 is believed to have caused a loss of bond, thereby influencing results of the O-cell tests and precluding any direct comparison. The third case involved STN and O-cell tests of shafts in Florida limestone. Paikowsky et al. stated that several factors, including highly variable site conditions and factors related to the tests, prevented a direct comparison of results.

FEM reported by Paikowsky et al. (2004b) suggests that differences in rock-socket response between O-cell testing and top-load testing may be affected by (1) modulus of the rock mass, E_M, and (2) interface friction angle, ϕ_i. Paikowsky first calibrated the FEM model to provide good agreement with the results of O-cell tests on full-scale rock-socketed shafts, including a test shaft socketed into shale in Wilsonville, Alabama, and a test shaft in claystone in Denver, Colorado, described by Abu-Hejleh et al. (2003). In the FEM, load was applied similarly to the field O-cell test; that is, loading from the bottom upward. The model was then used to predict behavior of the test shafts under a compression load applied at the top and compared with the equivalent top-load settlement curve determined from O-cell test results. Figure 67 shows a comparison of the top-load versus displacement curves for the Alabama test, one as calculated from the O-cell test and the other as predicted by FEM analysis. The curves show good agreement at small displacement (<0.1 in. or 2.5 mm); however, the curve derived from FEM analysis is much stiffer at higher displacement. This exercise suggests that the equivalent top-load settlement curve derived from an O-cell load test may underpredict side resistance for higher displacements; that is, the O-cell derived curve is conservative. Further FEM analyses reported by Paikowsky et al. (2004b) suggest that the differences between loading from the bottom (O-cell) and loading in compression from the top are the result of differing normal stress conditions at the interface, and that these differences become more significant with increasing rock mass modulus and increasing interface friction angle.

These numerical analyses suggest that differences in the response of rock sockets to O-cell test loading and top-down compression loading may warrant consideration in some cases. Ideally, side-by-side comparisons on identical test shafts constructed in the same manner and in rock with similar characteristics and properties are needed to assess differences in response. However, it is expected that the potential differences, if any, will eventually be identified and incorporated into interpretation methods for O-cell testing. In the meantime, the O-cell test is providing state transportation agencies with a practical and cost-effective tool for evaluating the performance of rock sockets and it is expected that the O-cell test will continue to be used extensively.

Instrumentation such as sister bars with strain gages makes it possible to better determine the load distribution and load transfer behavior during an O-cell load test. This information can then be used to make more refined predictions of load transfer behavior under head load conditions.

In summary, some of the advantages of the O-cell for axial load testing of rock-socketed shafts include:

- Ability to apply larger loads than any of the available methods (important for rock sockets) and

- With multiple cells or proper instrumentation, it can isolate socket base and side resistances from resistance of other geomaterial layers.

Limitations of the O-cell test for use by state DOTs include:

- Shaft to be tested must be predetermined, because it is not possible to test an existing shaft;
- For each installed device, test is limited to failure of one part of the shaft only;
- There are possible concerns using test shaft as a production shaft;
- Interpretation methods that account for differences in loading mode are not yet fully developed; and
- There are currently no ASTM or AASHTO standards specifically for O-cell load tests.

Interviews with state DOT engineers for this study show that the O-cell test has been an integral tool in advancing the understanding and use of rock-socketed drilled shafts. The Kansas DOT (KDOT) experience is representative of several other states. The following is based on an interview with Robert Henthorne, KDOT Chief Geologist. The geology of the western half of Kansas, located in the High Plains physiographic province, is dominated by thick sequences of sedimentary rocks, mostly sandstone, shale, and limestone. Until approximately 1995, virtually all highway bridges were founded on shallow foundations or H-piles driven to refusal on rock. Drilled shafts were not considered a viable alternative because of uncertainties associated with both design and construction. With encouragement from FHWA, KDOT engineers and geologists initiated a long-term program of training, education, and field load testing to better match foundation technologies with subsurface conditions. Workshops on drilled shaft design, construction, inspection, and nondestructive testing (NDT), sponsored by FHWA and the International Association of Foundation Drilling (ADSC), were conducted at the invitation of KDOT. KDOT began using drilled shafts as bridge foundations where appropriate. Several bridge sites in western Kansas were designed with rock-socketed shafts. To address the lack of experience with these conditions, O-cell testing was incorporated into the larger bridge projects. In almost every case, the O-cell test results showed side and base resistances considerably higher than the values used for preliminary sizing of the shafts, and valuable experience was gained with construction methods, effective cleanout strategies, NDT methods, etc. KDOT now has O-cell test results on rock-socketed shafts from nine projects and has developed in-house correlations between rock mass properties and design parameters for commonly encountered geological formations. Drilled shafts now comprise approximately 70% to 80% of new bridge foundations, and shaft designs are more economical because there is a high level of confidence in capacity predictions, based directly on the load tests.

The approach taken by KDOT illustrates how field load testing, in this case with the O-cell, can be incorporated into an overall program leading to increased use and improved design methods for rock-socketed foundations. The Colorado DOT has also used O-cell testing to improve its design procedures for rock-socketed shafts, as documented by Abu-Hejleh et al. (2003).

Statnamic

The STN load test was developed in the late 1980s by Berminghammer Foundation Equipment of Hamilton, Ontario. Its use in the U.S. transportation industry has been supported by FHWA through sponsorship of load testing programs, as well as tests conducted with an STN device owned by FHWA for research purposes.

In this test, load is applied to the top of a deep foundation by igniting a high-energy, fast-burning solid fuel within a pressure chamber. As the fuel pressure increases, a set of reaction masses is accelerated upward, generating a downward force on the foundation element equal to the product of the reaction mass and the acceleration. Loading occurs over a period of approximately 100 to 200 ms, followed by venting of the pressure to control the unloading cycle. Load applied to the foundation is monitored by a load cell and displacement is monitored with a photovoltaic laser sensor. The concept is illustrated schematically in Figure 68. STN equipment is available for test loads as high as 30 MN.

Processing of the load and displacement time histories is required to convert the STN measurements into an equivalent, static load-displacement curve. The analysis accounts for dynamic effects that may include damping and inertial effects. The unloading-point method as reported by Horvath et al. (1993) provides a relatively straightforward method for determining static resistance using measurements made at the top of the shaft during a STN test. Test interpretation is also discussed by Brown (1994) and El Naggar and Baldinelli (2000).

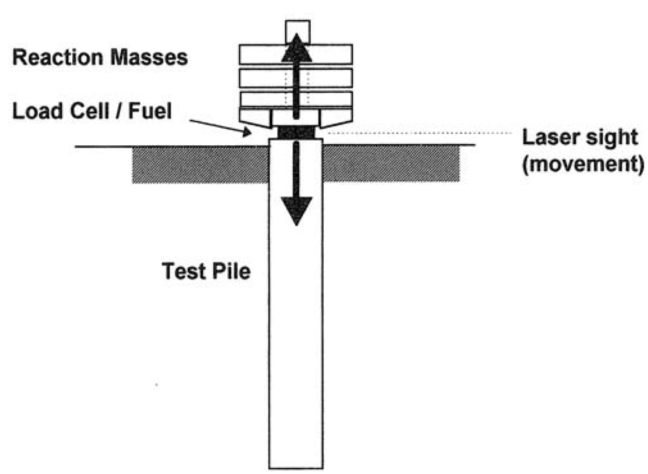


FIGURE 68 Schematic of STN load test (O'Neill et al. 1997).

Mullins et al. (2002) recently introduced the segmental unloading point method, which uses top and toe measurements as well as strain measurements from along the length of the foundation. The segmental unloading point method enables determination of load transfer along various segments of the foundation, an advantage for rock-socketed shafts to separate resistance developed in rock from that developed in portions of the shaft embedded in soil. The analysis is automated using software provided by the testing firm and equivalent static load-displacement graphs are produced immediately for evaluation. All data are stored for future analysis and reference.

During the 1990s, FHWA performed or funded STN and correlation studies with conventional static load tests to develop standardized testing procedures and data interpretation methods (Bermingham et al. 1994). Numerous other studies have further expanded the database of case histories and performance studies. The result is that STN testing is now a well-developed technology that is highly suitable for use by state DOTs for axial load testing of rock-socketed shafts. STN advantages identified by Brown (2000) include:

- Large load capacity, applied at top of shaft;
- Can test existing or production shaft;
- Economies of scale for multiple tests;
- Amenable to verification testing on production shafts; and
- Reaction system not needed.

Disadvantages include:

- Capacity high, but still limited (30 MN);
- Rapid loading method, as rate effects can be significant in some soils (less in rock);
- Mobilization costs for reaction weights; and
- Not currently addressed by ASTM or AASHTO standards.

Mullins, as reported in Paikowsky et al. (2004b), analyzed a database of 34 sites at which both STN and static load tests were conducted on deep foundations. The data included load tests on four drilled shafts in rock at two sites, one site each in Florida and Taiwan. The objective of the study was to develop recommendations for LRFD resistance factors when axial compression capacity is based on STN testing. The authors recommend a resistance factor of 0.74 for all deep foundation types in rock (not specific to drilled shafts) when tested by STN. In addition, a rate effect factor (REF) is recommended to account for rate effects when using STN results by the unloading point method. The REF varies with soil or rock type and recommendations are given here. If the segmental unloading point method is used (requiring strain gages), separate REF factors can be applied to each segment to account for different soil or rock types. This analysis addresses the disadvantage cited previously regarding rate effects.

Derived Static = REF*UP-derived capacity
REF = 0.96 for rock
0.91 for sand
0.69 for silt
0.65 for clay.

Dynamic Impact Testing

A dynamic compression load test can be carried out by dropping a heavy weight onto the head of the shaft from various heights. The shaft is instrumented with strain gages and accelerometers to measure the force and impact velocity of the stress wave generated by the dynamic impact. The measurements are correlated to driving resistance to predict load capacity. A review of various available drop weight systems and evaluation of the method is given by Paikowsky et al. (2004c). A typical drop weight system consists of four components: (1) a frame or guide for the drop weight, (2) the drop weight (ram), (3) a trip mechanism to release the ram, and (4) a striker plate or cushion, as shown in Figure 69. Various configurations of modular weights can be used to provide ram weights as high as 265 kN (Hussein et al. 2004) and drop heights are adjustable up to 5 m (Paikowsky et al. 2004c). A rule of thumb given by Hussein et al. is that a ram weight of 1% to 2% of the expected shaft capacity be available on site.

Drop weight load testing interpretation relies on analysis methods similar to those used in standard dynamic pile testing. Strain gage and accelerometer measurements at the top of the pile are used to evaluate characteristics of stress wave propagation. If sufficient shaft resistance is mobilized, it is possible in theory to relate the stress wave characteristics to shaft capacity using available PDA (Pile Driving Analyzer) technologies. Drop weight testing of drilled shafts has not been used extensively on bridge foundations in the United States, in part because other available methods (e.g., O-cell and STN) provide a more direct measurement of static resistance. According to DiMillio (1998), test results on FHWA projects have not demonstrated sufficiently good agreement between drop weight and other tests. The drop weight tests reportedly overpredicted measured capacities.

Drop weight testing for rock sockets is suitable for post-construction tests at bridge sites where questions arise during construction regarding the performance of as-built foundations. This application is illustrated by the case of the Lee Roy Selmon Crosstown Expressway, in Tampa, Florida. The columns supporting an elevated section of roadway are founded on drilled shafts socketed into limestone. During construction of the superstructure, one of the columns suddenly underwent more than 3 m (11 ft) of settlement as a result of the failure of the drilled shafts. Subsequent investigations determined that the failed shafts were not founded in sound limestone as believed, raising questions about the capacity of all 218 drilled shafts supporting the elevated roadway. As part of an investigation to determine how many

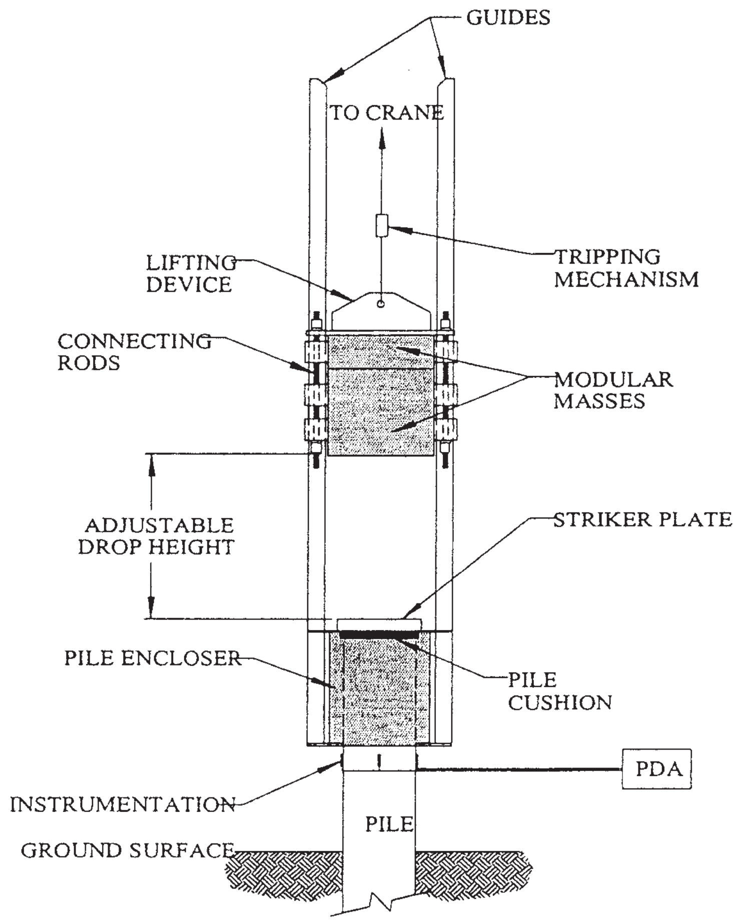


FIGURE 69 Schematic of drop weight system (Paikowsky et al. 2004c).

shafts might need remediation, dynamic load tests were conducted on 12 of the shafts supporting existing columns using the pile driving hammer shown in Figure 70. Testing proved the design capacity of 11 of the 12 shafts tested. This case also illustrates the need for thorough subsurface investigation when socketing into limestone. In this case, rock elevations were found to be highly variable. Seismic methods used in combination with borings in the post-failure investigation provided a more detailed geologic model of site conditions.

Interpretation Framework for Static Axial Load Tests

Carter and Kulhawy (1988) and Kulhawy and Carter (1992b) proposed a method for interpretation of static axial load tests on rock-socketed shafts. The method involves analyzing a static axial load-displacement curve from a load test according to the analytical closed-form solutions presented in chapter three (Eqs. 69–95). The parameters back-calculated from the load test could then be used to evaluate effects of various design parameters on the load-displacement behavior of trial designs that differ from that of the test shaft. The method is applicable to shafts that satisfy the criteria for rigid behavior, given as

$$\left[\frac{E_c / E_r}{2D / B}\right]^2 \geq 1 \tag{149}$$

in which E_c = modulus of the reinforced-concrete shaft, E_r = rock mass modulus, D = socket length, and B = socket diameter.

The analysis is applied to two cases: (1) shear socket under compression or uplift and (2) complete socket under compression. The shape of a load-displacement curve from a load test is modeled in terms of constant slopes (S), which are related mathematically to the model parameters described in chapter three. Consider the load-displacement curve for a shear socket loaded in compression, as shown in Figure 71. Three parameters are required to idealize the geometry of the curve. S_1 is the slope of the initial portion, S_2 is the approximated slope of the full-slip portion of the curve, and Q_i is the intercept on the vertical axis ($w_c = 0$) of the line with slope S_2. For a rigid shaft, the measured curve parameters

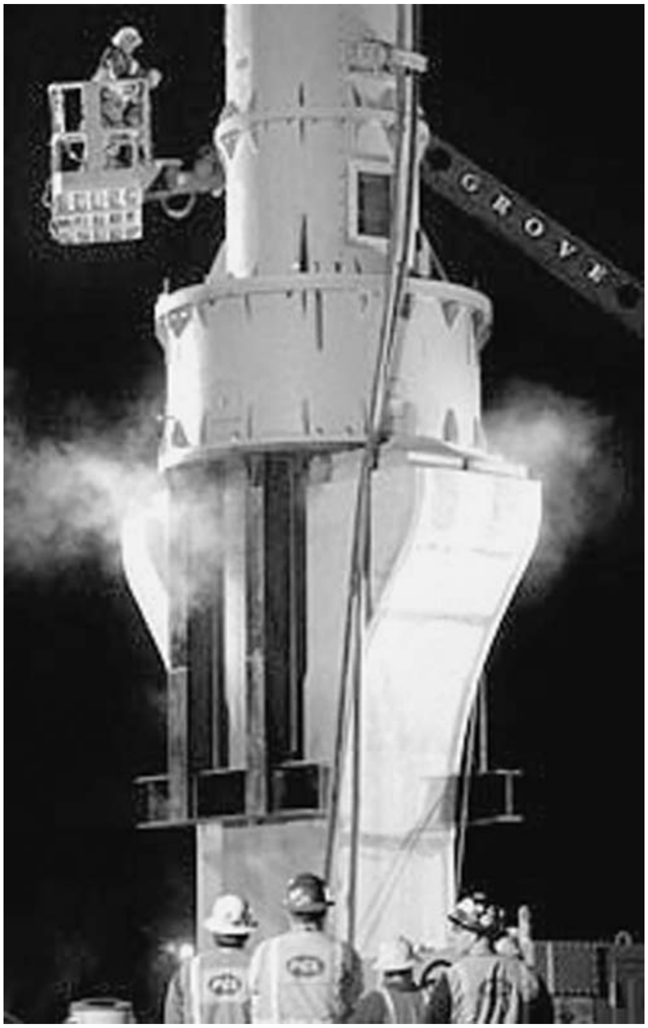


FIGURE 70 Dynamic load testing of shaft-supported column in Tampa, Florida.

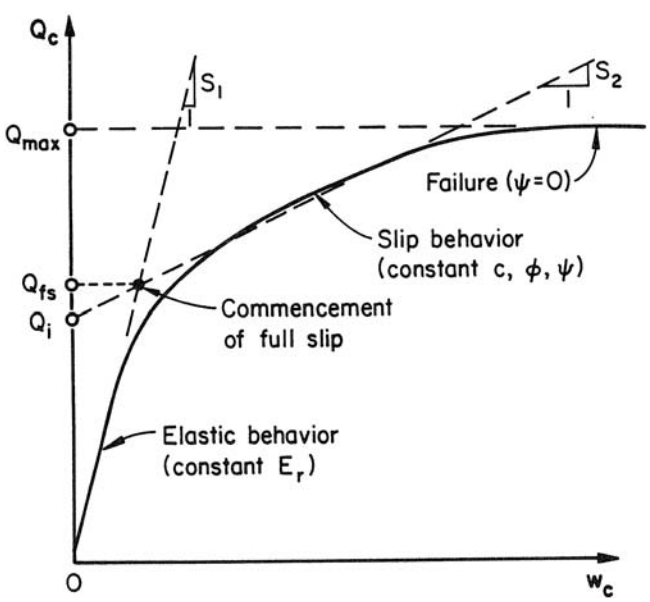


FIGURE 71 Interpretation of a side-shear-only test (Carter and Kulhawy 1988).

theoretically are related to the elastic model parameters by the relationships given here (a):

(a) Shear Socket — Compression or Uplift

$$E_r = \left[\frac{(1+\nu_r)\varsigma}{\pi D}\right] S_1$$

$$\tan\phi \cdot \tan\psi = \left(\frac{1}{2\varsigma}\right)\left(\frac{S_2}{S_1 - S_2}\right)$$

$$c = (2\varsigma \tan\phi \cdot \tan\psi + 1)\frac{Q_i}{\pi BD}$$

with

$$\varsigma = \ln\left[5(1-\nu_r)\frac{D}{B}\right] \tag{150}$$

in which ν_r = Poisson's ratio of the rock mass, ϕ = interface friction angle, and ψ = interface angle of dilation, and c = interface cohesion.

For a complete socket under compression in which the base load-displacement is determined (Figure 72), the load-displacement curve is approximated by S_1, S_2, Q_i, and S_3, the slope of the base load-displacement curve. The curve parameters are related to the elastic model parameters as given in (b),

(b) Complete Socket — Compression

$$E_r = \left[\frac{(1+\nu_r)\varsigma}{\pi D}\right](S_1 - S_3)$$

$$E_b = \left[\frac{(1+\nu_b{}^2)}{B}\right] S_3$$

$$\tan\phi \cdot \tan\psi = \left(\frac{1}{2\varsigma}\right)\left(\frac{S_2 - S_3}{S_1 - S_2}\right)$$

$$c = (2\varsigma \tan\phi \tan\psi + 1)\frac{Q_i}{\pi BD}$$

in which E_b = modulus of the rock mass beneath the shaft base.

Carter and Kulhawy (1988) applied the technique described to 25 axial load tests reported in the literature by back-calculating values of the model parameters E_r, E_b, c, and ($\tan\phi$ $\tan\psi$) from load-displacement curves using the equations given previously. A limitation of the model described earlier is that the assumption of rigidity may be less acceptable for shafts in harder rocks where the modulus values for the rock mass and the shaft material are closer. The reader is advised to review the original publications for further assumptions and derivations of the equations.

Lateral Load Testing

A significant number of states indicated in the questionnaire that lateral loading governs the design of rock-socketed

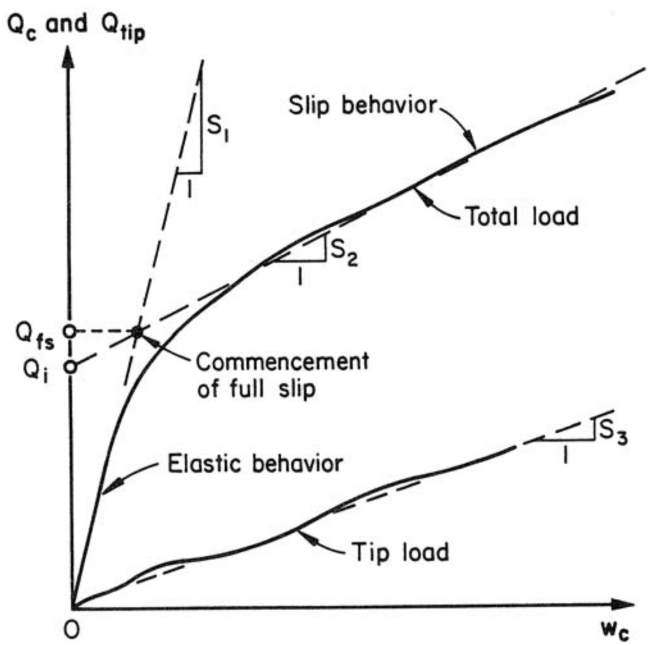


FIGURE 72 Interpretation of a complete socket test (Carter and Kulhawy 1988).

shafts for a significant percentage of projects (Question 25). However, as noted in chapter four, very few lateral load tests have been conducted on rock-socketed shafts. Methods for conducting lateral load tests on deep foundations include conventional methods, Osterberg load cell, and STN.

Conventional Lateral Load Test

The conventional method for conducting a lateral load test is given in ASTM D3966 and involves pushing or pulling the head of the test shaft against one or more reaction piles or shafts. A variety of arrangements for the test shaft and reaction shaft are possible and these are given in detail in Reese (1984) and Hirany and Kulhawy (1988). One approach is to use two shafts and apply the load such that both shafts are tested simultaneously, providing a comparison between two shafts. A load cell is used to measure the applied lateral load and dial gages or displacement transducers attached to a reference beam can be used to monitor lateral deformation. Thorough treatment of instrumentation for lateral load tests can be found in Reese (1984) and Hirany and Kulhawy (1988).

Drilled shafts are often used where the designer wishes to take advantage of their large lateral load capacity, especially that of large-diameter shafts. Analysis often shows that the geomaterials in the upper part of the ground profile have the most significant influence on lateral deformations and lateral load transfer. A critical part of lateral load testing is to have detailed knowledge of the site stratigraphy, particularly at the depths corresponding to approximately the first 10 diameters of the shaft. Other important points to consider when conducting conventional lateral load tests, as pointed out by O'Neill and Reese (1999) are summarized as follows.

The test site conditions and test shaft should be selected and built to match as closely as possible the actual conditions to which they will be applied. Items such as overburden stresses acting in the resisting soil and rock layers, groundwater and surface water conditions, shaft dimensions and reinforcing, and construction methods all can have a significant influence on the lateral load response of a drilled shaft. To the extent possible, these conditions should be matched by those of the load test.

Analysis of the load test results will be interpreted using the analytical methods presented in chapter four. The most widely used method is the *p-y* curve method, in which *p-y* curves are fit to obtain agreement with the load test measurements. As a minimum, it is therefore necessary to have reliable measurements of ground line shear load, ground line deflection, and rotation (requires two deflection points separated by a known vertical distance). To define *p*-y curves accurately over the length of the shaft requires measurements of the deflected shape of the shaft, which can be done using slope inclinometer measurements. A more accurate method to determine *p-y* curves (or to evaluate any analytical method) is to establish bending moment as a function of depth, which can be done by installing a steel tube with closely spaced strain gages along the length of the shaft. This approach is most appropriate for tests conducted for applied research; for example, to develop new methods for establishing *p-y* curves in rock.

Boundary conditions must be considered carefully when back-fitting analytical models and then applying the model for design. In a lateral load test, the boundary conditions at the head of the shaft will normally be free of any rotational restraint and have zero applied moment and zero axial load. Service boundary head conditions are likely to include some head restraint and possibly axial load and moment. Also, the nonlinear moment–*EI* relationships must be accounted for both in the load test and in the analysis.

Four states (California, Massachusetts, New Jersey, and North Carolina) reported the use of conventional lateral load tests on rock-socketed shafts. Although lateral load testing is not as common as axial testing, conventional testing has been the method of choice for lateral. Other methods have, so far, been used on a limited basis. These include lateral O-cell tests (at least two states, South Carolina and Minnesota) and lateral STN (Alabama, Florida, Kentucky, North Carolina, South Carolina, and Utah). Several states that did not respond to the survey are known to have conducted lateral STN (Ohio and Virginia).

Lateral Osterberg Load Test

The O-cell can be embedded in a drilled shaft and oriented such that the load is applied in the horizontal direction. The method is described by O'Neill et al. (1997) for a case in which the Minnesota DOT required representative *p-y* curves for a stratum of friable sandstone situated beneath a thick layer of normally consolidated clay. Shafts socketed into the sandstone were to support a bridge undergoing ice loading. The test was conducted at a nearby location in which a 26.7 MN O-cell was positioned vertically within a 1.22-m-diameter socket, as depicted in Figure 73, and used to thrust the two halves of the socket against the rock. Lateral force and deflection measurements were used to derive *p-y* curves. The authors point out that care must be taken in interpreting the results, because the stress–strain conditions created by the test are not the same as in a laterally loaded socketed shaft that is loaded at its head and not split.

Lateral O-cell testing of rock sockets offers some of the same advantages as for axial O-cell load testing, namely the elimination of a structural loading system at the ground level. Also, the test provides the ability to apply lateral loading at predetermined depths, such as within the rock socket. Further research is needed to establish guidelines for proper procedures and to define correct analyses that account for the differences in boundary conditions, load transfer, and soil and rock resistance, compared with a shaft loaded at its head. It is also worth noting that the lateral split socket test may provide a means to measure the in situ rock mass modulus of deformation (E_M).

Lateral Statnamic

The STN load test has also been adapted for lateral loading. The device is mounted on steel skids supported on the ground allowing the reaction masses to slide on rails, as shown in Figure 74. The lateral STN test can simulate lateral impact loading such as might occur against a bridge pier from a vessel.

The lateral STN test can also be used to derive the static lateral response, but requires appropriate instrumentation and correct analysis of the test results. In tests described by Brown (2000), the following instrumentation was used:

- Load cell,
- Displacement transducers,
- Accelerometers on top of cap or shaft,
- Downhole motion sensors,
- Resistance-type strain gages, and
- Megadac Data Acquisition System.

Figure 75*a* shows the measured dynamic response of the shaft in terms of force, acceleration, and lateral displacements versus time. The curves showing measured lateral displacement from three measurements are identical and cannot be distinguished in the figure. Dynamic response is separated into static, inertial, and damping components. A *p-y* analysis (using LPILE or FBPIER) is fit to obtain a reasonable match between the measured load-displacement response for each component of force (static, inertial, and damping). Load versus displacement curves derived are shown in Figure 75*b* based on analysis of the dynamic response in Figure 75*a*.

The lateral STN test is reported as to be safe, controlled, and economical. Its principal advantage lies in the ability to measure directly the dynamic lateral response and to provide a derived static response. This test is a valuable tool for the design of bridge foundations to withstand dynamic lateral loading from earthquakes, wind, and vessel impacts. The test may also be used in place of a conventional static test. Lateral loads up to 18 MN may be possible.

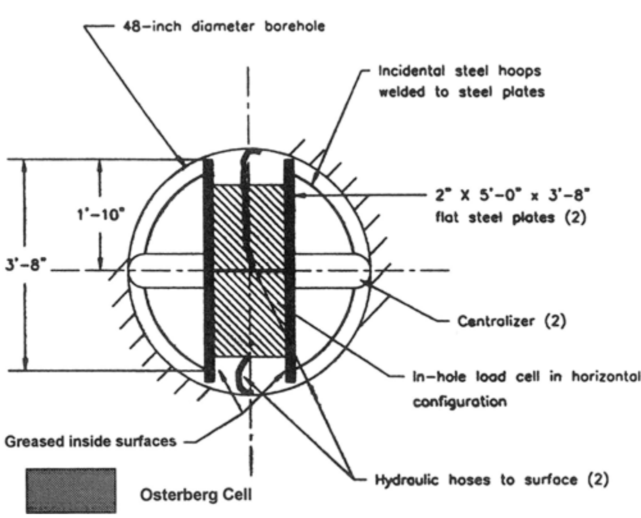
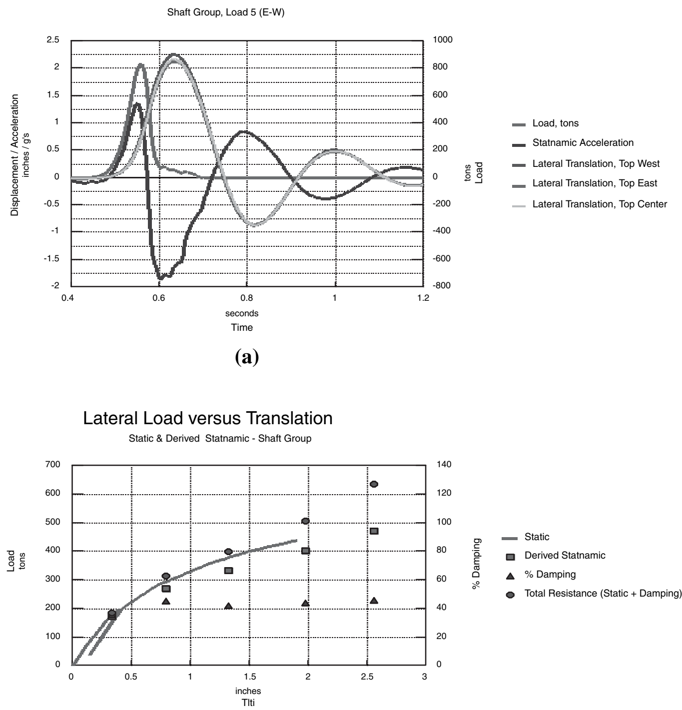


FIGURE 73 Top view of O-cell arrangement for lateral split socket test (O'Neill 1997).

FIGURE 74 Lateral STN load test (*Courtesy*: L. Fontaine).



(b)

FIGURE 75 Results of lateral STN test: (*a*) measured dynamic response; (*b*) derived static response.

CONSTRUCTABILITY, INSPECTION, AND QUALITY ASSURANCE

These topics are considered together because they encompass activities having a single objective: construction of a high-quality, rock-socketed drilled shaft foundation that performs in accordance with the design assumptions. As illustrated in the flow chart diagram of Figure 3, chapter one, the final design is based on input from three general sources: (1) site characterization, (2) geotechnical analysis, and (3) structural analysis and modeling. Plans and specifications are developed that reflect generally accepted practices based on the collective experience of the construction and engineering communities. Examples of model specifications include those given in Chapter 15 of the FHWA *Drilled Shaft Manual* (O'Neill and Reese 1999), ACI *Standard Specification for the Construction of Drilled Piers*, ACI 336.1-98 (1998), and specifications developed by state and federal transportation agencies with extensive experience in drilled shaft use. In addition, effective specifications will address issues that are unique to the specific conditions that determine the final design, including constructability issues which, ideally, are accounted for in all three of the input categories identified previously. In the following paragraphs, these topics are discussed individually, but in practice they must be integrated into the design concepts discussed in this synthesis.

Constructability

Much emphasis has been placed on constructability of drilled shafts by FHWA and through efforts of the International Association of Foundation Drilling (ADSC). The FHWA *Drilled Shaft Manual* (O'Neill and Reese 1999) addresses constructability and its role in drilled shaft design. The manual also forms the basis of a National Highway Institute (NHI) course on drilled shafts that is available through FHWA. A separate NHI course that certifies inspectors for drilled shaft construction (Williams et al. 2002) also has a strong emphasis on constructability and was developed with significant contractor input. ADSC provides short courses, workshops, and a library of publications focused on construction-related issues for drilled shafts. ADSC also provides "constructability reviews" of individual projects in which independent contractors review the project plans and specifications and offer advice on its constructability. This step could be incorporated into the overall process depicted in Figure 3, as denoted in the flow chart by "constructability review."

Integrating constructability into a drilled shaft project involves taking a common sense approach to design that accounts for the methods, tools, and equipment used by contractors to build the shafts. No attempt will be made here to identify all of these issues, but items identified by the survey and that relate specifically to rock sockets are discussed.

Schmertmann et al. (1998) and Brown (2004) present guidelines for ensuring quality in drilled shaft construction and some recent advances in materials that have applications in both soil and rock. The key elements to be considered to avoid the most commonly observed construction problems are:

- Workability of concrete for the duration of the pour;
- Compatibility of congested rebar and concrete;
- Control of stability of the hole during excavation and concrete placement, especially with casing;
- Proper consideration and control of hydrostatic balance and seepage;
- Bottom cleaning techniques and inspection; and
- Drilling fluid that avoids contamination of the bond between concrete and bearing material or excessive suspended sediment.

New developments in concrete mix design, in particular mixes described as self-consolidating concrete (SCC), can provide benefits for drilled shaft construction. The characteristic of SCC that is most beneficial is very high slump flow. Reinforcement cages with a high density of steel bars, often necessary especially for seismic design, make it difficult to provide the necessary clear spacing between bars that will ensure flow of concrete to the outside of the cage. The flow properties of SCC have been shown to reduce potential defects associated with incomplete cover or voids caused by inadequate flow of concrete.

Prompt placement of concrete is another construction practice that promotes quality in the as-built shaft. Delay in concrete placement increases the potential for slump loss and, in some cases, has been identified as a cause of reduced side resistance (Schmertmann et al. 1998).

Several states identified problematic construction issues when the slurry method of construction is used in rock sockets. One issue is whether slurry has a detrimental effect on side resistance of rock sockets. Thirteen states indicated that they restrict the use of slurry in rock sockets and one state expressed "concerns with use of drilling fluids instead of casing." In many situations, if casing is used to support the hole, the need to use slurry is eliminated. Typically, casing need only extend to the top of rock if the rock-socket portion of the hole will remain open without caving. If there is water in the overburden, the casing can be sealed into the rock, dewatered, and the socket can then be excavated without support. However, there are situations where a contractor may deem it necessary to introduce slurry. For example, when rock is highly fractured it may not be possible to seal the casing sufficiently to prevent water inflow, and a contractor may elect to use slurry. In this case, slurry may be used to balance the hydraulic head to prevent seepage into the hole that can disturb the material at the base of the shaft, an issue related directly to design decisions on whether to include base resistance in the design. For reverse circulation drilling, slurry may be used as the circulating fluid (e.g., the Richmond–San Rafael Bridge shown in Figure 56).

There are few data showing the effects of properly mixed and handled slurry on rock-socket side or base resistances. Slurry that does not possess the appropriate viscosity, density, and sediment content, or that is allowed to remain in the hole (and not agitated) long enough to form a thick filter cake, will almost certainly reduce side resistance compared with a shaft drilled and poured under dry conditions, in either soil or rock. However, if sound practices are followed by an experienced contractor and there is proper inspection, slurry drilling for rock sockets can be an effective construction method, assuming the slurry is handled in a manner that avoids contamination of the interface bond or excessive suspended sediment.

In certain rock types, there is evidence that use of polymer slurry may be beneficial to rock-socket side resistance. The Kentucky DOT requires polymer slurry for drilling in rock that exhibits low values of slake durability index. Typically, this is the case in certain shale formations in Kentucky. Slaking occurs when the shale is exposed to water, and can cause formation of a smear zone, reducing side resistance considerably, as demonstrated by Hassan and O'Neill (1997). Apparently, the polymer slurry prevents softening and the resulting smear zone, although there have not been load tests in which a direct comparison has been made. This issue deserves further research.

One state DOT identified the following as a problematic construction issue: "various methods used to force a dry pour," indicating that some measures taken to avoid placing concrete under water or slurry are more detrimental than allowing a wet pour. Both Schmertmann et al. (1998) and Brown (2004) describe a case that seems to contradict some commonly held ideas about casing versus wet hole construction of rock sockets. A drilled shaft installed through 12 m of soil and socketed into rock was constructed using a full-length casing (to provide downhole visual inspection). A load test using the Osterberg load cell indicated a mobilized side resistance in the socket of 0.5 MN, much less than expected. A second shaft was constructed, but using a wet hole method with tremie placement of concrete and without casing into the rock. Load testing of this shaft indicated more than 10 MN of side resistance in the socket. The difference is attributed to a decrease in concrete workability during the time required to remove the casing after concrete placement, preventing formation of a good bond along the socket interface. Trapping of debris between the casing and rock could also have occurred and may have smeared cuttings along the sidewalls. The lesson of this case is that the construction method should be selected to provide the best product for the given conditions, and that in many situations a wet hole method is the most effective and will not adversely affect shaft behavior if done properly. Forcing a dry pour may cause more problems than it solves.

Another good reason to review the ground conditions carefully before allowing "dry hole" construction is identified by Schmertmann et al. (1998). If the groundwater elevation is above the base of the hole, dry conditions inside the socket result in a hydraulic gradient causing inward seepage as illustrated in Figure 76. They describe several cases where seepage degraded side resistance and base resistance. Maintaining a slurry or water level inside the hole sufficient to balance the groundwater pressure eliminates the inward gradient

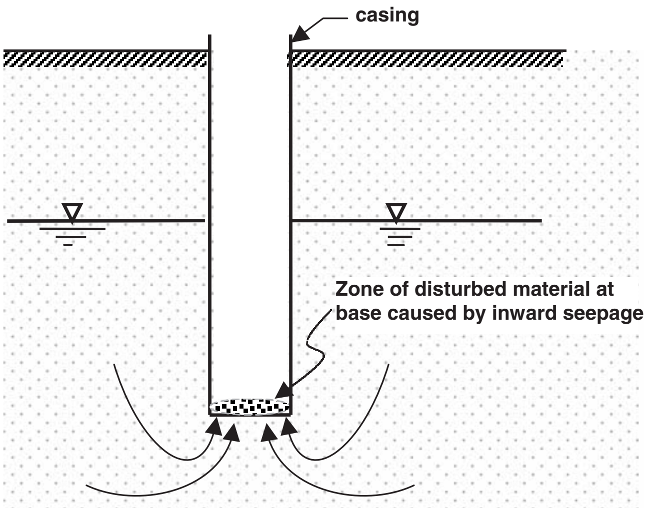


FIGURE 76 Development of disturbed base caused by high seepage gradient toward bottom of a cased hole.

and prevents base and side disturbance. The authors cite several cases in which comparisons of Osterberg load cell test results on shafts poured both wet and dry show this effect.

The most common factor cited in construction claims associated with rock-socketed shafts is "differing site conditions," that is, the subsurface conditions actually encountered during construction are claimed to be materially different from those shown in boring logs. Responses to the questionnaire did not indicate that claims were a major obstacle to the use of drilled shafts for most states. However, one state DOT gave the following response when asked to comment on issues "pertaining to the use of rock-socketed drilled shafts by your agency" (Question 6): "Most result in claims due to the requirement to include 'Differing Site Conditions' on all contracts."

The same agency responded as follows to Question 36 pertaining to perceptions of construction problems:

> We design for low bidding contractors to get the contract and the construction problems that will result. Rock may be harder than the contractor thought when bidding and planning the job. Thus the drilling equipment brought out is often unable to drill or very slow to drill the rock. This results in costly contractor claims.

Claims for differing site conditions are part of the geotechnical construction field, but measures can be taken to minimize them. For example, one contractor interviewed for this study noted that geotechnical reports often place strong emphasis on rock of the *lowest* strength, because these layers may control side or base resistances for design. However, for estimating drilling costs, contractors need information on rock layers of the *highest* strength, because that will dictate the type of drilling and tools needed to bid the job accurately and to carry out the construction properly. Transportation agencies might consider surveying contractors to find out exactly what information contained in their boring logs is most helpful for bidding on rock-socket jobs, and what additional information could improve their ability to perform the work. Another contractor interviewed for this study stated that the rock classification system of the ISRM is useful to determine what type of tool (rock auger, core barrel, or downhole hammer) will be most effective. The ISRM system places rock into one of seven categories (R0 through R6) based on strength, as described in chapter two (see *Rock Material Descriptors*).

An issue identified by several states is the discrepancy that sometimes occurs between the elevation corresponding to top of rock as shown in boring logs and as encountered during construction. The Washington State DOT uses language in their special provision for rock sockets that reportedly works well and is summarized as follows. For shafts with a specified minimum penetration into the bearing layer and no specified base elevation, the contractor furnishes each reinforcing cage 20% longer than specified in the plans. The increased length is added to the bottom of the cage. The contractor then trims the reinforcing cage to the proper length before placement. The DOT assumes the cost of the excess steel, but believes that cost is offset by avoiding construction delays, disputes, and claims that may occur otherwise.

Other specific issues identified by states in the questionnaire pertain to inadequate cleanout buckets, improper placement of concrete with pump trucks, and a case in which temporary casing to support the overburden with the same diameter as the rock socket resulted in the casing being dragged down into the socket, requiring additional socket drilling. There is a constructability lesson in each of these cases.

Certain geologic conditions are associated with more challenging construction and may require more detailed investigation and flexibility in the approach to construction. Some of the more notorious of these include: (1) karstic conditions associated with limestone and other rocks susceptible to solution, (2) rock with steeply dipping discontinuities, (3) well-developed residual soil deposits grading into partially weathered rock and then unweathered bedrock, (4) alternating hard and soft layers of rock, and (5) glacial till. Each of these conditions presents its own unique set of construction challenges and different approaches are required to address them successfully. A question that often arises in some of these environments is "what is rock?," or perhaps more importantly, "what is not rock?" On some projects, certain geomaterials may be rock for pay purposes, but not for design. If these issues can be addressed before construction and there is good communication between owners and contractors, a reasonable approach that results in a successful project can usually be developed. When the difficulties are not anticipated but are encountered during construction, the likelihood of claims and disputes is much higher. Drilling of a trial installation shaft (also referred to as a "method" or "technique" shaft) before bid letting can identify many of the problems that will be encountered during production drilling and should be considered whenever there are major questions about the subsurface conditions and what is required to construct rock sockets successfully.

Inspection and Quality Assurance

Inspection is the primary method for assuring quality in the construction of drilled shafts. The philosophy and methods of drilled shaft inspection are covered in Chapter 16 of the FHWA *Drilled Shaft Manual* (O'Neill and Reese 1999) and are the subject of a video and a *Drilled Shaft Inspector's Manual* (Baker 1988) available from the ADSC. A certification course for drilled shaft inspectors is offered by the NHI of FHWA, and a Participants Manual was developed as part of the course (Williams et al. 2002). Table 21 is a partial listing of inspection issues pertaining specifically to rock-socket construction.

Special emphasis is required in making a strong connection between drilled shaft design and inspection. Practically, this involves providing inspection personnel with the knowledge and tools required to verify that drilled shafts are constructed and tested in accordance with the design intent. The starting point for inspection personnel is to have a thorough understanding of (1) subsurface conditions, (2) the intent of the design, and (3) how items 1 and 2 are related. The inspector's sources of information for subsurface conditions include the geotechnical report, boring logs, and communication with the design engineer. For rock sockets, inspectors should be trained to understand the information presented in boring logs pertaining to rock. This includes being familiar with the site and geomaterial characterization methods described in chapter two. Inspectors require basic training in rock identification, testing, and classification, and should be familiar with rock coring procedures, the meaning of RQD, compressive strength of intact rock, and terminology for describing characteristics of discontinuities, degree of weathering, etc. Inspectors should be aware of design issues such as whether the shaft is designed for side resistance, base resistance, or lateral resistance, and in which rock layers the various components of resistance are derived.

Before construction, inspectors should know how the contractor plans to construct the shafts. This requires knowledge of the tools and methods used for construction in rock. A valuable aid is the Drilled Shaft Installation Plan, a document describing in detail the contractor's tools and methods of construction. O'Neill and Reese (1999) describe the minimum requirements of an installation plan and recommend that it be a required submittal by the contractor.

A fundamental design issue is the degree to which the rock mass over the depth of the socket coincides with the conditions assumed for design. Therefore, some type of

TABLE 21
INSPECTION ITEMS FOR ROCK SOCKETS

Inspection Responsibility	Primary Items to Be Addressed	Required Skills or Tools
Knowledge of site conditions	Rock types, depths, thicknesses, engineering properties (strength, RQD); groundwater conditions	Competency in rock identification and classification; ability to read and interpret core logs
Knowledge of design issues	Rock units providing side, base, and lateral resistances Design parameters: shaft locations, socket depths and diameters, reinforcement details	Basic understanding of design philosophy for drilled shafts under axial and lateral loading Familiarity with standard specifications, plans, special provisions, shop drawings, and contractor submittals
Knowledge of contractor's plan for socket construction	Rock excavation tools (augers, coring, hammers, other) and methods (e.g., casing, slurry) Classification of rock for pay purposes	Review of Drilled Shaft Installation Plan
Observations and record keeping during socket excavation	Identification and logging of excavated rock Tools used by contractor for each geomaterial (tool description, diameter, rate of excavation) Occurrence of obstructions, removal method Depth to top of rock Sidewall conditions (roughness, smearing) Roughening or grooving of sidewalls Use and handling of slurry and casing Inspection methods and devices (e.g., SID) Coring at the base Cleanout specs., verification method	Competency in field identification of geomaterials; Appropriate forms*, including: Rock/Soil Excavation Log Rock Core Log Inspection Log Construction and Pay Summary
Sampling and testing	Sampling of rock for lab tests; Field tests on rock; e.g., point load, hardness; NDT/NDE	Proper sampling/testing equipment and knowledge of procedures

*See Williams et al. (2002) for descriptions of inspection forms.
Notes: RQD = rock quality designation; SID = shaft inspection device; NDT = nondestructive testing; NDE = nondestructive evaluation.

downhole inspection is needed. Responses to Question 11 of the survey reveal a wide variety of methods used for this purpose. Nine states reported that coring is required into rock below the bottom of the shaft after the excavation to base elevation is complete. Typical required depths range from 1.5 m to 10 m, three diameters, etc., although one state requires coring 15 m below the bottom of the shaft. Coring below the base during construction allows a determination to be made of the adequacy of rock below the base to (1) provide the base resistance assumed in the design; (2) ensure that the base is bearing on bedrock and not an isolated boulder ("floater"); and (3) detect the presence of seams, voids, or other features that would require changes in the base elevation or other remedial actions.

Five states reported using a probing tool to inspect core holes at the bottom of the completed excavation (Figure 77). This method, which in most cases requires downhole entry by the inspector, is most useful for detecting seams of soft material in discontinuities. It is most applicable in limestone and dolomite where the bedrock surface is highly weathered, irregular, and filled with slots and seams of clayey soil. Proper safety measures are paramount for downhole entry. Five states reported using fiber optic cameras for inspection of core holes, which is safer and provides visual evidence of seams, cavities, and fractures, but does not provide the "feel" of probing that may be useful in karstic formations. Four of the states reporting use of probe rods are in the Southeast where karstic conditions are most common.

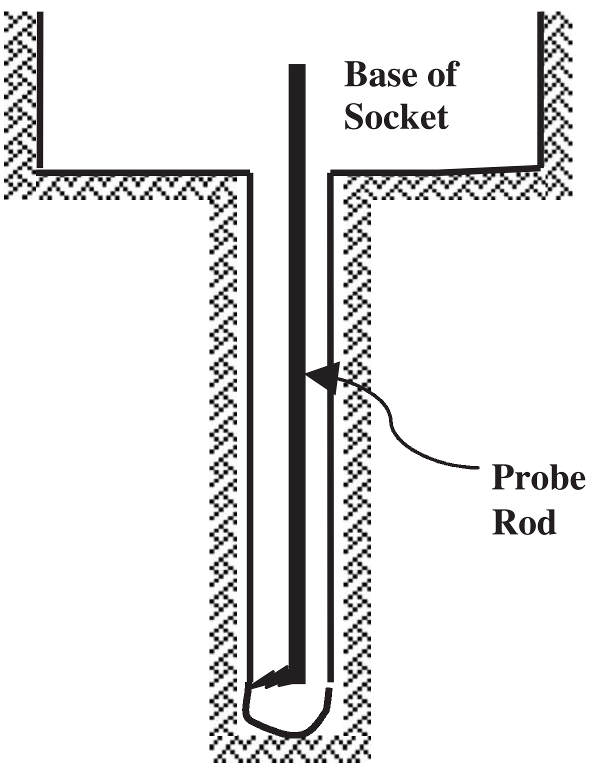


FIGURE 77 Rock probing tool (after Brown 1990).

Most states include specifications for conditions at the bottom of the hole that must be satisfied before pouring concrete. Some distinguish between shafts designed for base resistance and those designed under the assumption of zero base resistance. A very typical specification (five states) is "minimum 50% of the base area to have less than 12 mm (0.5 in.) and maximum depth not to exceed 38 mm (1.5 in)." Some states allow up to 300 mm (6 in.) of loose material when base resistance is neglected.

When sockets are poured under dry conditions, common inspection methods to verify bottom conditions are either visual inspection or downhole cameras. For wet pours (under slurry or water) the most common method is to lower a weighted tape (e.g., a piece of rebar on the end of a tape measure) to the bottom of the hole and "feel" the bottom conditions by bobbing the weight against the bottom. Although somewhat subjective, an experienced inspector can differentiate between clean water or slurry and contaminated conditions. Downhole cameras are available that permit viewing of conditions under water or slurry. A device used by the Florida DOT referred to as a shaft inspection device or SID has been used successfully in slurry shafts (Crapps 1986). The device, shown in Figure 78, has a color television cam-

FIGURE 78 Shaft inspection device or SID.

era encased in a watertight bell and equipped with a light source and a water jet for clearing sediment to provide clear pictures of the shaft sides and base. The SID was developed in Australia specifically for inspection of rock sockets under bentonite slurry. North Carolina also reported using a SID, and several other states use downhole cameras to inspect sockets under water or slurry.

The survey shows that some states neglect socket-base resistance altogether if concrete is placed under slurry or water (Question 14). The rationale is that base conditions cannot be verified with sufficient reliability to be sure that a poor base, or "soft bottom," condition is avoided. This refers to a layer of disturbed soil, slurry, or contaminated concrete at the base, which may allow excessively large downward movement before the resistance of the underlying rock can be mobilized. These concerns may be justified under some conditions. However, as described in chapter three (see Table 16 and Figure 23), there are good reasons to account for base resistance even for shafts constructed under wet-hole conditions. Construction and inspection practices that can be taken to avoid poor base conditions include appropriate specifications and quality control on properties of slurry at the bottom of the hole prior to concrete placement, cleanout of slurry contaminated with cuttings or suspended particles before concrete placement, use of a weighted tape to "feel" the bottom of the hole as an inspection tool, downhole viewing devices for inspection of bottom conditions (e.g., SID), and proper use of a pig or other device in the tremie pipe to prevent mixing of concrete and slurry. Post-grouting of the shaft base is a measure that could be incorporated into design and construction to provide quality base conditions in drilled shafts.

It is instructive to observe that most states that have incorporated field load testing of rock sockets into their foundation programs, using a method that allows measurement of base load-displacement, now include both side and base resistances in their design calculations. This is based on load test results that show, when proper quality control is applied, that base resistance is a significant component of shaft resistance at service loads.

Nondestructive Testing and Evaluation

Field tests to evaluate the integrity of as-built drilled shafts are now used widely in the industry as part of overall quality assurance. Nondestructive methods for testing (NDT) and evaluation (NDE) are covered in Chapter 17 of the *Drilled Shaft Manual* (O'Neill and Reese 1999) and in several other publications. The survey for this study included a question asking respondents to identify any issues pertaining to NDT and NDE that are unique or important specifically for rock-socketed drill shafts. No issues were identified, other than the need to consider locating NDT access tubes in the reinforcing cage so that the entire assembly is able to fit into a socket that may be of a smaller diameter than the shaft above the socket.

EXAMPLES OF DIFFICULT GEOLOGIC CONDITIONS

Some of the most difficult conditions for drilled shaft construction and inspection are karstic limestone and residual profiles that grade from soil to weathered rock to intact rock. Experiences and approaches to these conditions identified by the literature review are summarized here.

Shafts in Limestone

Use of drilled shafts in karstic terrain is considered by Knott et al. (1993), Sowers (1994), and others. Brown (1990) describes design and construction challenges of using drilled shafts in hard pinnacled limestones and dolomites encountered in the Valley and Ridge and Cumberland Plateau physiographic provinces. Subsurface conditions are highly irregular owing to extensive weathering. Although intact rock strengths may be high (up to 70 MPa or 10,000 psi), numerous seams, slots, and cavities are typically filled with residual clayey soils (see Figure 79). Boulders and chert nodules are often embedded in the soils. Drilling through soil is often performed in the dry soil and then a casing set when rock is encountered. Drilling in the rock is difficult and can involve a combination of rock augers, drill and shoot methods, and core barrels. Sudden groundwater inflow is common upon encountering soil seams and slots.

In this environment of extreme variability the actual soil and rock conditions for a specific drilled shaft cannot be determined with any degree of accuracy before construction. Design, construction, and inspection have to be flexible enough to adjust to conditions actually encountered. For example, where shafts can be shown to bear at least partially on sound rock, base resistance is assumed, but highly

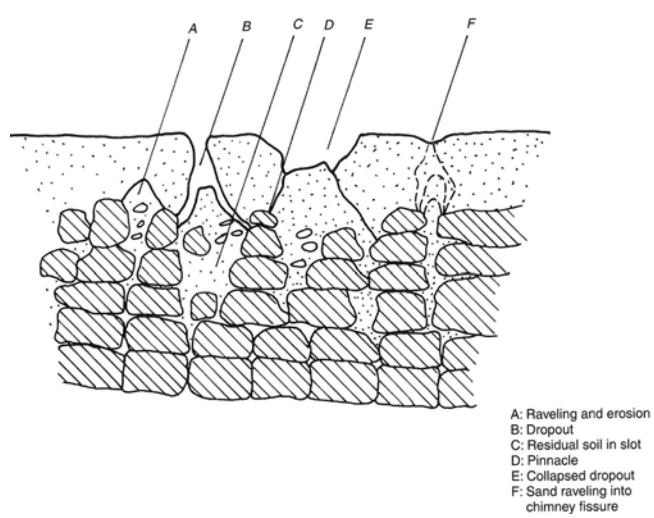


FIGURE 79 Features of karstic terrain (Knott et al. 1993).

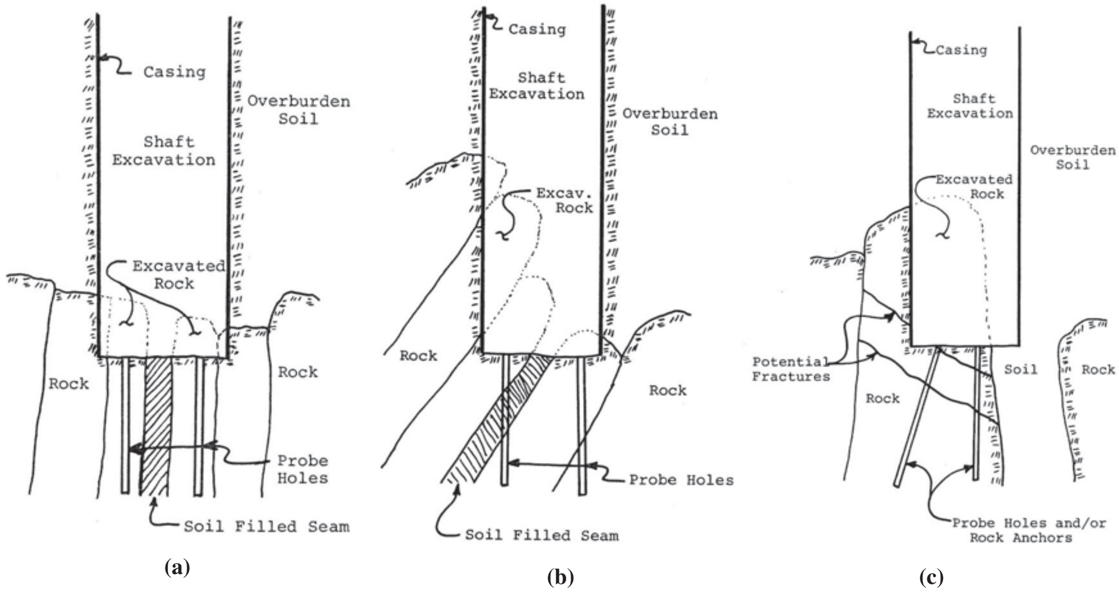


FIGURE 80 Commonly encountered conditions for shafts in pinnacled limestone (Brown 1990).

conservative values are used to account for the presence of seams at the base. This case is illustrated in Figure 80*a*, in which a probe rod placed down one or more probe holes drilled into the base can be used to determine the extent and nature of the seam. One criterion for acceptance is rock coverage of 75% or more of the base area and vertical seams. Figure 80*b* shows a nonvertical seam, which should be detectable by one of the probe holes and might necessitate additional drilling to preclude shear failure along the seam. Alternatively, the seam could be excavated and grouted. This technique would not be recommended if seepage is expected into the excavated seam. Where shafts are bearing on a section of rock bounded by vertical seams or slots and the possibility of fracturing exists, rock anchors are sometimes used to transfer load across potential fracture planes, as illustrated in Figure 80*c*. Rock anchors or micropiles are also used to transfer load across horizontal seams filled with soft soil and detected by probing beneath the base.

To provide the flexibility needed for design, inspection, and construction, creative contracting approaches are also needed. Brown (1990) reported that contracting such work on a unit cost basis provided the flexibility needed to deal with the unknown quantities of soil versus rock drilling, concrete overpours, rock anchoring, drilling of probe holes, etc. The engineer estimates the unit quantities, but actual payment is based on unit costs of material quantities actually used. This requires careful inspection and record keeping.

Drilled Shafts in the Piedmont

The Piedmont Physiographic Province of the eastern United States, extending from Alabama to New Jersey, is characterized by decomposed metamorphic rocks and a weathering profile characterized by unpredictable variability in the thickness and quality of the weathered materials. Drilled shafts are used extensively for major structures in this region, primarily because it has been recognized that large axial loads can be supported if a shaft is extended to either decomposed or intact rock.

Gardner (1987) identified three general weathering horizons in the Piedmont: (1) residual soil, representing advanced chemical alteration of the parent rock; (2) highly altered and leached soil-like material (saprolite) retaining some of the structure of the parent rock; and (3) decomposed rock (locally referred to as partially weathered rock), which is less altered but can usually be abraded to sand- and silt-sized particles. The underlying intact rock is typically fractured near its surface but increases in quality with depth. The thickness and characteristics of each zone vary considerably throughout the region and may vary over short horizontal distances, and boundaries between the horizons may not be distinct. Figure 81 shows a typical profile based on borings at one site. Factors that make drilled shafts challenging to design and build in the Piedmont are:

- Highly variable subsurface profiles,
- Presence of cobbles and boulders,
- Steeply dipping bedrock surfaces, and
- Difficulties in distinguishing between soil, partially weathered rock, and intact rock for pay purposes.

The first of these makes it difficult to determine ahead of time what the final base elevation will be for shafts required to reach intact rock. At least one boring at each drilled shaft location can help to address this issue. Figure 82 illustrates two conditions that can cause “refusal” before the shaft

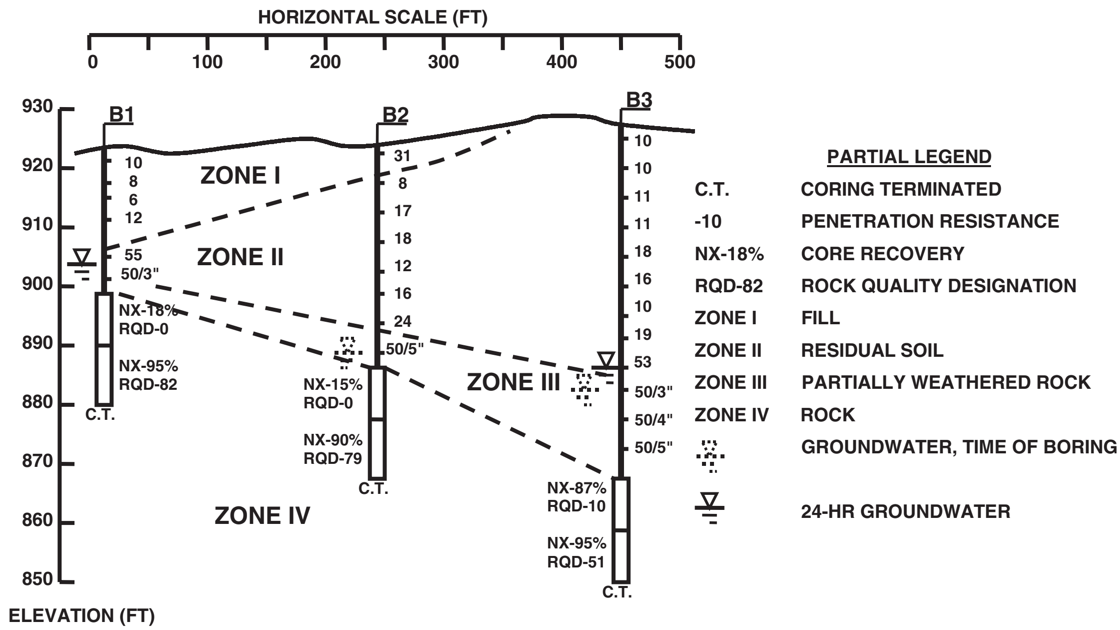


FIGURE 81 Typical Piedmont subsurface profile (after Schwartz 1987).

is drilled to its design base elevation. When refusal is encountered on a boulder that is "floating," questions may arise concerning whether the boulder is an obstruction or constitutes drilling in rock. Similarly, when sloping bedrock is first encountered, the volume of material excavated to reach base elevation may be disputed as to whether it is soil or rock, and drilling into sloping rock can be difficult. One approach is to install casing until one edge of the casing hits rock, then drill a smaller diameter pilot hole into the rock followed by drilling to the design diameter and advancement of the casing.

Gardner (1987) reviews design methods for axial loading of drilled shafts in Piedmont profiles, including recommendations for design side and base resistances in rock and methods used to determine relative load transfer between side and base. Harris and Mayne (1994) describe load tests in Piedmont residual soils. O'Neill et al. (1996) used the tests of Harris and Mayne to develop the recommendations for side resistance in cohesionless IGM from Standard Penetration Test results, as presented in chapter three. Both Gardner (1987) and Schwartz (1987) outline measures that can be taken to minimize construction delays and contract disputes when building rock-socketed shafts in Piedmont profiles. The principal requirements are: (1) thorough site investigation, (2) design and construction provisions that can accommodate the unpredictable variations in subsurface materials and final base elevations, and (3) construction specifications and contract documents that facilitate field changes in construction methods and shaft lengths. Successful construction also depends on highly qualified inspectors and clear communication between design engineers, contractors, and inspectors.

These examples illustrate the challenges that can be encountered in the design and construction of rock-socketed drilled shafts as a result of certain geologic conditions, as well as approaches that others have found successful for addressing such challenges. Every foundation site is unique geologically, and successful design and construction approaches are those that are adapted to fit the ground conditions. Mother Nature is quite unforgiving to those who behave otherwise.

SUMMARY

Construction and issues related to constructability are integral parts of drilled shaft foundation engineering. A review of rock drilling technologies is presented and shows that a wide variety of equipment and tools is available to contractors for building drilled shafts in rock. The design, manufacturing, and implementation of rock drilling tools is a field unto itself and it is important for foundation designers to be knowledgeable about the availability and capability of tools and drilling machines. Constructability issues are interrelated with all of the steps shown in the flowchart of Figure 3, depicting the design and construction process for rock-socketed shafts. Beginning with site characterization and continuing through final inspection, constructability is taken into account in foundation selection, in design methods through the effects of construction on side resistance, in critical design decisions such as whether base resistance will be included, in writing of specifications pertaining to use of slurry and bottom cleanout, and in matching inspection tools and procedures to construction methods. The literature review identified many aspects of constructability pertaining to rock

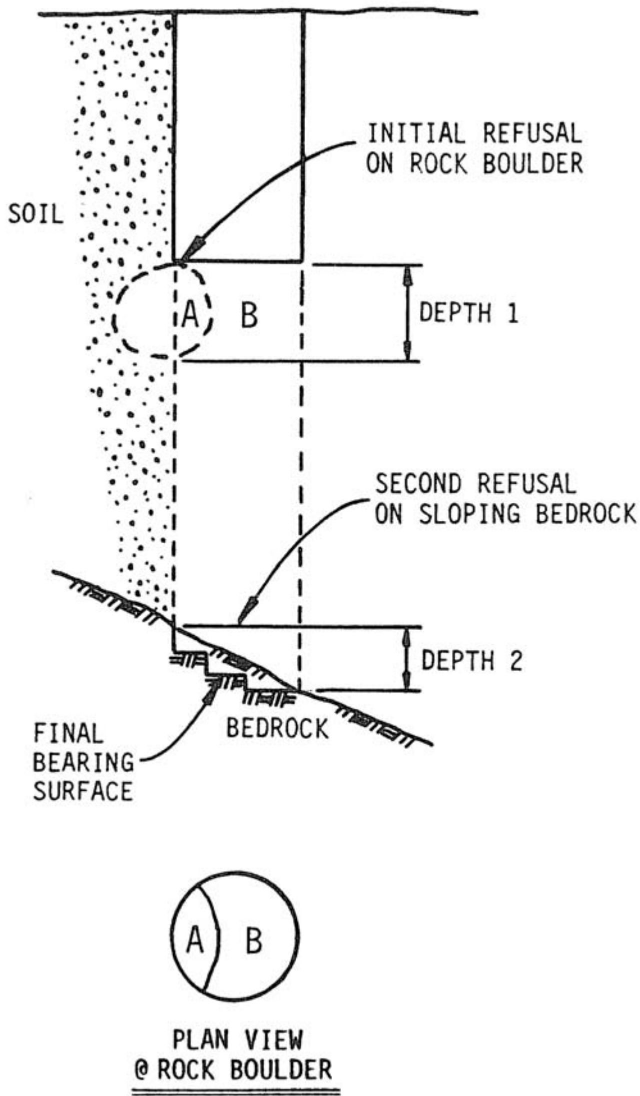


FIGURE 82 Typical drilling in Piedmont soils and rock (Schwartz 1987).

sockets and these are summarized. Practices that can improve constructability; for example, the use of SCC and installation of method shafts are identified.

Field load testing of rock sockets has increased since the advent of innovative load testing methods, especially the O-cell and the STN. The basic mechanics of these and other tests are described, followed by a review of current applications of each to testing of rock-socketed shafts. The survey shows that many states are using the O-cell to verify, and also to improve, design methods of rock sockets. A description of the KDOT experience with O-cell testing in rock is presented as an example. Load testing is also shown to be a factor in increased use of rock-socketed drilled shafts by transportation agencies. Finally, load testing with the O-cell has been a useful tool for identifying and evaluating poor versus good construction practices. The report by Schmertmann et al. (1998), referenced several times in this chapter, is a particularly useful source for that information.

Inspection and field quality control are recognized in the drilled shaft industry as the critical link between design and construction. Excellent sources of information on inspection are available and these are identified. The NHI inspector certification course is highly recommended for all inspection personnel. Some of the tools identified by the survey and literature review that can be most effective for rock-socket inspection are the SID, coring of rock beneath the socket-base, use of probing tools, and downhole fiber optic cameras.

Two geologic environments in which rock-socket construction poses special challenges, karstic limestone and Piedmont residual profiles, are presented to illustrate some of the practices that lead to successful projects. Matching of design and construction strategies to ground conditions is the essence of constructability.

CHAPTER SIX

CONCLUSIONS

This synthesis identified technologies and practices available to transportation agencies for utilization of rock-socketed drilled shafts as reliable and cost-effective structure foundations. All thirty-two of the state transportation agencies responding to the survey are currently using rock-socketed shafts, some quite extensively (more than 20 projects per year). The single Canadian agency responding to the survey has not used drilled shafts extensively to date. Use of rock-socketed drilled shafts for transportation structures has increased significantly over the past 25 years and technologies applied to design, construction, and testing have advanced considerably.

The design process for structural foundations by state transportation agencies is outlined in chapter one. Responsibilities typically are separated into geotechnical and structural categories. Site characterization, geomaterial property evaluation, and design issues related to geotechnical capacity or load-deformation analysis are normally addressed as geotechnical issues, whereas structural modeling and reinforced-concrete design are normally carried out by structural engineers. Design for lateral loading requires significant input and analysis by both geotechnical and structural personnel.

The overall process of design and construction (i.e., engineering) is shown to consist of highly interrelated factors, requiring an integrated approach to drilled shaft foundations. Figure 3 in chapter one illustrates the process in the form of a flowchart. Adequate site characterization is needed to obtain the basic information required for both geotechnical analysis and construction. Constructability issues are best addressed during the design process, when decisions such as whether to include side resistance, base resistance, or both must be made on the basis of anticipated subsurface conditions, construction methods, load testing, inspection methods, and experience.

SITE AND GEOMATERIAL CHARACTERIZATION

The most valuable and reliable information for rock-socket design is obtained by drilling and taking core samples of the rock at the location of each structural foundation. Careful logging of rock core, photographic records, and proper handling of core to obtain samples for laboratory testing provide the basic information that will be used for rock mass classification, evaluation of engineering properties of intact rock and rock mass, and baseline information needed to assess constructability. Drilling also provides the means to conduct in situ tests. Every transportation agency that responded to the survey currently relies on rock coring as the primary source of design and construction information for rock-socketed shafts.

Geophysical methods can provide additional valuable information when applied appropriately by competent users. *NCHRP Synthesis 357: Use of Geophysics for Transportation Projects* (Sirles) identifies the major geophysical methods that are applicable to geotechnical investigations and found that overall use of geophysics by transportation agencies is expanding. Seismic refraction for establishing depth to bedrock is the most common use of geophysics for drilled shafts in rock. However, of 33 responding agencies, only 8 (24%) reported using geophysics, including 7 that use seismic refraction and 1 that uses electrical resistivity. These data suggest that geophysical methods are not used widely for investigations related specifically to foundations in rock. Survey results from the Sirles study show that agency experience is mixed, with both successful and unsuccessful cases being cited. Factors associated with successful cases (for depth to bedrock) are: sufficient number of borings to validate and correlate the seismic results, interpretation by qualified geophysicists, and clear understanding of the capabilities and limitations of the technology.

Geophysical methods that show potential for rock site investigations include electrical resistivity tomographic profiling and borehole televiewers. Multi-array resistivity methods have shown the ability to provide accurate images of subsurface profiles in karstic terrains when used in conjunction with borings. Borehole televiewers, both acoustic and optical, may have limited applicability to rock foundations. They are primarily useful for providing detailed information on structural discontinuities. For large or critical rock-socket projects, where the orientation and condition of discontinuities in situ is a critical concern, these devices can supplement information obtained from more conventional core logging. Other potentially useful methods are downhole seismic and crosshole seismic. A case described by LaFronz et al. in 2004, "Geologic Characterization of Bridge Foundations, Colorado River Bridge, Hoover Dam Bypass Project," showed good correlation between rock mass modulus from downhole seismic testing and rock mass modulus from correlations to the Geological Strength Index (GSI). This approach warrants further consideration.

The laboratory test most widely applied to foundation design is uniaxial compression of intact rock. Properties obtained are uniaxial compressive strength (q_u) and elastic modulus of intact rock (E_R). Poisson's ratio may also be determined. Uniaxial compressive strength is used directly in the most widely applied design methods for evaluating unit side resistance, unit base resistance, and limiting pressure under lateral loading. Modulus of intact rock is not used directly, but rather with other rock mass characteristics to evaluate rock mass deformation modulus (E_M). Other laboratory tests applicable to rock-socket design include the splitting tensile strength (used for side resistance in limestone) and the point load strength (an index of compressive strength). Direct shear testing is used to assess shear strength of rock mass discontinuities and can be used to test shear strength of rock/concrete interfaces. Slake durability is used to assess potential for rapid degradation and smearing of weak rocks during construction of rock sockets.

Rock mass classification systems have useful applications in foundation design. The Rock Mass Rating (RMR) as given by Bieniawski in *Engineering Rock Mass Classifications* (1989) incorporates the most important rock mass characteristics (including rock quality designation) that control the strength and deformability of a rock mass. The RMR is useful as an overall indicator of rock quality and suitability as a founding material, and is the basis for correlations to rock mass strength and modulus. Approximately one-half of the states responding to the survey reported using RMR in connection with rock-socket projects. The GSI introduced by Hoek et al. in *Support of Underground Excavations in Hard Rock* (1995) can be evaluated on the basis of RMR and is also correlated directly with rock mass strength, through the Hoek–Brown strength criterion, and rock mass modulus of deformation. GSI is now being used in geomechanical models for bearing capacity in rock and for evaluation of limiting lateral pressure for shafts in rock under lateral loading.

In situ testing in rock is used primarily to obtain rock mass modulus (E_M). Pressuremeter (PMT) and borehole jack are the methods being used. Modulus values obtained by PMT are affected by the scale of the test relative to the scale of rock mass features (discontinuity spacing and orientation) and may or may not be representative for the purpose of foundation analysis. The principal use of rock mass modulus is in analyzing axial and lateral load deformation response of rock sockets. There are several published *p-y* curve criteria for laterally loaded shafts that incorporate modulus as determined by PMT. The issue of whether the modulus values from PMT are the most appropriate requires further research. Table 15 in chapter two summarizes the rock mass properties required for design of rock-socketed shafts.

DESIGN FOR AXIAL LOAD

Methods for predicting the behavior of rock sockets under axial loading have developed considerably since the 1970s. The literature review showed that axial load transfer is reasonably well understood in terms of its basic mechanisms. Effects of interface roughness, socket-length-to-diameter ratio, modulus ratio, and other variables have been studied analytically and experimentally, providing a broad understanding of the underlying concepts. Although design methods do not incorporate all of the governing parameters explicitly, understanding the underlying mechanics is useful in many ways, including to provide a framework for understanding the limitations of empirical design methods.

Specific methods for predicting side and base capacities must be in a form that matches the level of knowledge of the ground conditions and that is based on commonly measured rock and intermediate geomaterials (IGM) properties. Chapter three of this synthesis provides a summary and review of available methods and it is shown that conservative, reliable, first-order estimates can be made for design values of side resistance on the basis of uniaxial strength of intact rock. Geomaterial-specific methods are presented for Florida limestone, residual Piedmont soils (cohesionless IGMs), and weak argillaceous rocks (cohesive IGMs). A method based on direct correlation to Texas Cone Penetration Test results illustrates how some agencies use in-house methods.

For base capacity, a variety of methods have been proposed in the literature. Because several modes of failure are possible depending on structural characteristics of the rock mass, no single equation is applicable to all conditions. Furthermore, few studies have been conducted comparing proposed bearing capacity models with measured base capacities on socketed shafts loaded to failure. A 1998 study by Zhang and Einstein provided a first-order approximation of unit base resistance from uniaxial strength of intact rock, based on a limited database of field load tests. For intact rock, a conservative upper-bound unit base resistance can be taken as 2.5 times the uniaxial compressive strength. Two methods given in the *Canadian Foundation Engineering Manual* and incorporated into current AASHTO specifications are recommended for horizontally jointed sedimentary rocks. For highly fractured rock masses, a lower-bound estimate of ultimate bearing capacity can be made in terms of RMR or GSI.

Analytical methods for predicting axial load-displacement of rock sockets are needed to design shafts to limit settlement and to determine the percentage of load carried by base resistance under service load conditions. Methods based on elastic and elastoplastic finite-element modeling are available in the form of charts. Although these methods are useful, they are cumbersome. Simple closed-form solutions that are implemented easily on a spreadsheet are presented. Both approaches require knowledge of the rock mass modulus.

DESIGN FOR LATERAL LOAD

Methods for analysis of rock sockets under lateral loading are readily available to foundation designers, but currently are subject to uncertainties regarding their reliability. The survey shows that all states currently use the *p-y* method of analysis. Criteria for *p-y* curves in rock have been proposed and these are the most widely used at the current time. However, the proposed criteria were described as "interim" when they first appeared, because of the insufficient field load test data available for validation. Research aimed at improving *p-y* curve criteria for rock has been described. The proposed methods also require additional verification by comparisons with field load testing. A major feature of *p-y* methods of analysis is that they provide structural analysis of the reinforced-concrete shaft that incorporates the nonlinear moment–*EI* relationship. This feature provides a useful interface between geotechnical and structural design.

Analysis methods based on elastic continuum theory have been developed for lateral loading. The Carter and Kulhawy method (1992) requires a minimal number of parameters and is easy to implement by hand or spreadsheet, but is applicable only over the range of elastic response. The Zhang et al. method (2000) provides the complete nonlinear response, but requires more input parameters and relies on a finite-difference computer solution. These methods may be useful in the *Preliminary Foundation Design* phase (Figure 2, chapter one), for making first-order assessments of trial designs to satisfy service limit state criteria for lateral displacements. They are most applicable when the ground profile can be idealized as consisting of one or two homogeneous layers; for example, soil over rock.

LOAD TESTING

A positive development for drilled shaft design has been the introduction of several innovative field load testing methods. The Osterberg Cell (O-cell) and Statnamic (STN) tests can be conducted in less time, at lower cost, and with less equipment than conventional axial load testing methods. This has given transportation agencies the option of incorporating load testing into the design process on individual projects and developing databases of shaft performance in specific geologic environments. The experience of the Kansas Department of Transportation is described as a model example for incorporating O-cell testing into a comprehensive program that has resulted in more efficient use and design of rock-socketed shafts. Many of the states surveyed have taken advantage of O-cell and STN testing and this has resulted in a significant increase in load test data. It is suggested that a database of load test results be developed, analyzed, maintained, and made available to the wider research community.

The survey shows that states using the O-cell for axial load testing are less likely to neglect base resistance for design of rock-socketed shafts. O-cell testing almost always demonstrates that base resistance provides a significant portion of total axial resistance under service load conditions. Data from Crapps and Schmertmann in Figure 23, chapter three, show direct evidence of significant base load transfer when appropriate construction and inspection methods are applied to base conditions. Furthermore, O-cell and STN testing often result in higher values of allowable side resistance than would be calculated using the recommended prediction equations, which are intended to be conservative.

Lateral load testing of rock sockets can be conducted using O-cell and STN methods. The STN method may be particularly applicable for design of shafts subject to dynamic lateral loading, such as impact and seismic. Lateral O-cell testing has been demonstrated successfully, although research is suggested to develop procedures to relate lateral O-cell test results to *p-y* curve criteria and to parameters used in other analytical methods. Conventional static lateral load testing is still the most common method and is a proven approach to verifying performance and studying load transfer behavior. Lateral load testing on instrumented shafts is the only reliable method for validating *p-y* curves for design.

CONSTRUCTABILITY AND INSPECTION

Issues of constructability and inspection are related directly to rock-socket design and performance. Load testing, especially with O-cell methods, has helped to identify the effects of various construction methods on rock-socket performance. For example, the perception that construction of rock sockets is best facilitated by using full-depth casing and taking measures to permit a "dry" pour has been shown to have detrimental effects on side and base resistances. Use of water or slurry, when subjected to appropriate quality control, provides better performance by eliminating inward seepage, trapping of cuttings behind casing, and potential for smearing as the casing is removed.

Tools available for incorporating constructability into rock-socket design through specifications, plans, and inspection procedures are identified in several publications, including the FHWA *Drilled Shaft Manual* and the Participants Manual for the National Highway Institute Inspectors Certification Course. Several state agencies have developed model drilled shaft specifications that incorporate proven constructability practices (see for example, Washington State DOT *Geotechnical Design Manual* 2005). Recent developments in concrete mix design, such as self-consolidating concrete, are expected to provide improved constructability. Inspection tools such as the shaft inspection device used by Florida and North Carolina have direct implications for design. By providing a means to verify base conditions under water or slurry construction, designers are better prepared to include base resistance in socket design. Construction of "technique" or "method" shafts and contractor constructability reviews before publication of the final design and bidding phases are additional tools for incorporating constructability.

RESEARCH NEEDED TO ADVANCE STATE OF PRACTICE

Information gathered for this study suggests that development of improved practices for design and construction of rock-socketed drilled shafts might be achieved through the following research or wider dissemination of existing information.

Site Characterization

- Studies are needed to better define the best methods for determination of rock mass deformation modulus specifically for use in rock-socket design. In situ methods, including borehole jack and PMT, may yield different results and both could be compared with the most up-to-date correlations with RMR and GSI.
- A survey of contractors could be conducted to identify the rock mass information most useful for evaluating construction in rock; avoiding overemphasis on "weakest" materials.
- Application of geophysical methods to rock-socket design requires further research and development. Guidelines are needed for matching appropriate methods to site conditions. Case histories of successes and limitations could be published and distributed.
- Research is needed relating rock drillability to rock mass characteristics; correlations to RMR or GSI warrant investigation.
- Relationships between the reliability of rock and IGM engineering properties and resistance factors used in load and resistance factor design could be investigated and quantified sufficiently to support the resistance values recommended in AASHTO specifications; this topic could be the subject of ongoing research.

Design for Axial Loading

Sufficient analytical tools exist for the reliable design of sockets under axial loading. However, much of this information is widely scattered in the literature. The FHWA *Drilled Shaft Manual* and the 2006 Interim AASHTO *LRFD Bridge Design Specifications* include some available methods, but are not concise and clear in the presentation, and include some out-of-date methods. Numerous equations are presented in the literature for estimating base resistance of drilled shafts in rock. Surprisingly, very little data are available by which proposed methods can be evaluated. Studies are needed involving field axial load testing in which rock mass properties are well-documented and design equations for base resistance can be evaluated. Equations for incorporating roughness as a design parameter for unit side resistance have been proposed by several researchers. These design methods are limited because roughness is not a commonly measured parameter in the field. Construction techniques are constantly under development and innovative methods that can lead to improved quality should be encouraged and, where appropriate, developed further through research.

- Consider developing a manual or design circular focused specifically on drilled shafts in rock.
- Research is needed for axial load tests on instrumented shafts for the purpose of evaluating prediction equations for base resistance in rock; O-cell and STN tests are ideal for this purpose.
- Identify efficient and inexpensive field roughness measurement methods that can be incorporated into design equations; correlate roughness parameters to rock type, drilling tools, groundwater conditions, etc.
- Investigate the potential of base grouting as a quality assurance tool for rock-socketed shafts.

Design for Lateral Loading

Methods developed for analysis of deep foundations in soil, especially the *p-y* curve method, are the methods of choice for laterally loaded rock sockets. The principal limitation lies in the lack of proven *p-y* curve criteria for rock and IGM. This problem could be addressed by first conducting a comprehensive analysis of all existing load test results to evaluate proposed models, followed by research involving additional field load testing against which *p-y* curve criteria can be evaluated and calibrated.

- Conduct research to collect and analyze all existing lateral load test results, with the goal of establishing uniform criteria for *p-y* curve development.
- Transportation agencies could undertake research involving lateral load tests on properly instrumented rock-socketed shafts, designed specifically for testing and calibration of *p-y* criteria for rock and IGM.

Structural Design

Structural issues of concern to foundation designers, as identified by the survey, included uncertainty regarding apparently high shearing forces in shafts analyzed using *p-y* curve analyses and questions pertaining to moment capacity of short, rigid sockets. These issues can best be addressed by rigorous analytical methods in conjunction with load tests on carefully instrumented shafts in rock. A structural issue that has yet to be investigated as it pertains to deep foundations is the effect of confining stress on the strength and stiffness of reinforced concrete. It may be that concrete strength could be significantly increased under confining stresses typically encountered over the subsurface depths of many bridge foundations. More economical structural designs may be possible if this issue is investigated and applied in practice. Permanent steel casing contributes to the structural capacity of drilled shafts. Design methods that account explicitly for the steel casing are lacking in current design codes.

- Consider fundamental research with the goal of quantifying the effects of geologic confining stress on reinforced-concrete shear, moment, and compression behavior. Incorporate the results into structural design of drilled shafts.
- Conduct research and development of methods that account for permanent steel casing in the structural design of drilled shafts.

Management of Load Test Data

Large amounts of data from load tests on rock-socketed shafts, conducted for the purpose of research or for specific transportation projects, have been acquired, especially since the development of new testing methods. These data can be used most effectively if they are made available from a single source and organized in a systematic manner.

- Investigate placing those into a national database of load test results for rock-socketed drilled shafts, for use by transportation agencies and researchers.

EQUATION SYMBOLS

ENGLISH LETTERS—UPPERCASE

A_b = cross-sectional area of the foundation base.
A_c = cross-sectional area of concrete inside of spiral steel.
A_g = gross cross-sectional area of concrete shaft.
A_s = surface area of the side of the foundation.
A_{st} = total cross-sectional area of longitudinal reinforcement.
A_v = area of concrete in the cross section that is effective in resisting shear.
A_{vs} = cross-sectional area of shear reinforcement.
B = foundation diameter.
B_{ls} = diameter of a circle passing through the center of the longitudinal reinforcement.
C = correlation factor relating point load strength to uniaxial compressive strength of rock; correlation coefficient relating uniaxial compressive strength to ultimate unit side resistance.
D = foundation depth; distance between point loads in point load test; rock-socket diameter.
D_s = depth of embedment in rock; thickness of soil layer overlying rock.
E = elastic modulus; modulus of deformation.
E_b = elastic modulus of rock mass below the shaft base.
E_c = elastic modulus of the concrete shaft.
E_d = small-strain dynamic modulus.
E_e = effective elastic modulus of concrete shaft.
E_{ir} = initial elastic modulus of rock mass.
E_M = rock mass modulus of deformation.
E_p = elastic modulus of the pile (shaft) material.
E_r = elastic modulus of intact rock; modulus of rock mass above the base.
E_R = elastic modulus of intact rock.
E_s = elastic modulus of the shaft.
G^* = equivalent shear modulus of rock mass.
G_r = shear modulus of the elastic rock mass.
GSI = Geological Strength Index
H = horizontal load acting on a drilled shaft.
I_D = slake durability index.
IGM = intermediate geomaterial.
I_s = uncorrected point load strength index; moment of inertia of reinforced-concrete shaft.
$I_{s(50)}$ = point load strength corrected to a diameter of 50 mm.
I_ρ = dimensionless influence factor for elastic deformation.
J = coefficient used to evaluate ultimate lateral resistance in soil; bearing capacity correction factor that depends on the ratio of horizontal discontinuity spacing to socket diameter.
J_a = joint alteration number.
J_n = joint set number.
J_r = joint roughness number.
J_w = joint water reduction factor.
K = normal stiffness of rock-concrete interface.
K_b = socket depth factor.
K_{ir} = initial slope of *p-y* curve.
K_o = in situ coefficient of lateral earth pressure.
K_p = coefficient of passive earth pressure.
L = socket length.
$L_{\delta M}$ = length of equivalent fixed-end column considering lateral deflection owing to moment.
$L_{\delta V}$ = length of equivalent fixed-end column considering lateral deflection owing to shear.
$L_{\theta M}$ = length of equivalent fixed-end column considering rotation owing to moment.
$L_{\theta V}$ = length of equivalent fixed-end column considering rotation owing to shear.
L_s = nominal socket length.
L_t = total travel distance along socket wall profile for roughness determination.
M = bending moment.
N = standard penetration test value.
N_{60} = corrected N for field procedures corresponding to 60% hammer efficiency.
N_c = bearing capacity factor.
N_{cr} = bearing capacity factor.
N_q = bearing capacity factor.
N_γ = bearing capacity factor.
N_ϕ = bearing capacity factor.
OCR = overconsolidation ratio.
P = axial load acting on a drilled shaft; load at rupture in a point load test.
P_r = factored axial resistance.
P_u = factored axial load.
P_z = axial load on a deep foundation.
Q = Tunneling Quality Index.
Q' = modified Tunneling Quality Index.
Q_b = load transmitted to the base of a rock socket; ultimate resistance at the socket base.
Q_c = compressive force applied to the top of a drilled shaft.
Q_i = intercept on the vertical axis ($w_c = 0$) of axial load-displacement curve.
Q_{OC} = Osterberg cell test load.
Q_s = total side resistance (force).
Q_t = total compressive load applied to the top of the shaft.
REC = average percent recovery of rock core.
RF = roughness factor.
RMR = rock mass rating.
RQD = rock quality designation.
SRF = stress reduction factor.
S = joint spacing.

S_1 = slope of the initial portion of axial load-displacement curve.
S_2 = slope of the full-slip portion of axial load-displacement curve.
V_c = nominal shear resistance provided by concrete.
V_n = nominal shear resistance of reinforced concrete.
V_p = compressional wave velocity.
V_r = factored shear resistance of reinforced concrete.
V_s = shear wave velocity; nominal shear resistance provided by transverse steel.

ENGLISH LETTERS—LOWERCASE

a = empirical constant in Hoek–Brown strength criterion for rock mass.
b = empirical factor to account for effect of roughness on side resistance.
c = rock mass cohesion; soil cohesion.
c_{peak} = peak interface adhesion.
c_{residual} = residual interface adhesion.
c_u = undrained shear strength.
d = effective shear depth of reinforced concrete.
d_s = socket diameter.
f = shear wave frequency (hertz).
f_c' = compressive strength of concrete at 28 days.
f_{des} = design unit side resistance.
f_{su} = ultimate unit side resistance.
f_y = yield strength of reinforcing steel.
k_{ir} = dimensionless constant used in p-y curve criterion.
k_{rm} = constant used to establish the overall stiffness of a p-y curve.
m_b = empirical constant in Hoek–Brown strength criterion for rock mass.
n = ratio of rock mass modulus to uniaxial compressive strength of intact rock.
p = lateral soil or rock reaction per unit length of foundation.
p_1 = limit pressure determined from a pressuremeter test.
p_a = atmospheric pressure.
p_A = horizontal active earth pressure.
p_L = limit normal stress.
p_o = at-rest total horizontal stress.
p_{ur} = rock mass ultimate lateral resistance.
p_{ult} = ultimate lateral resistance of soil or rock mass.
q_u = uniaxial compressive strength of intact rock.
q_{ult} = ultimate bearing capacity.
q_t = split tensile strength.
r = radius of drilled shaft.
r_s = nominal socket radius.
s = empirical constant in Hoek–Brown strength criterion for rock mass; vertical spacing of the ties or pitch of the spiral for shear reinforcement.
s_c, s_γ, s_q = shape factors used in bearing capacity analysis.
s_v = vertical spacing between discontinuities.
t_d = aperture (thickness) of discontinuities.
u = lateral displacement at the groundline of socketed shaft.
w = distributed load along the length of the shaft.
w_c = axial displacement at the top of a socketed shaft.
x_r = depth below rock surface.
y = lateral deflection of a deep foundation.
z = depth below rock mass surface.
z_y = depth of yielding in soil and/or rock mass.

GREEK SYMBOLS—UPPERCASE

Δr = dilation or increase in shaft radius.
Δr = mean roughness height.
Δr_h = average height of asperities.

GREEK SYMBOLS—LOWERCASE

α = ratio of design to ultimate unit side resistance; ratio of rock mass modulus to modulus of intact rock; empirical adhesion factor relating unit side resistance to uniaxial strength of intact rock.
γ = total unit weight of rock or soil.
γ' = effective unit weight of rock or soil.
δ_r = relative shear displacement between concrete and rock.
δ_M = lateral deflection owing to moment.
δ_V = lateral deflection owing to shear.
ε = strain.
υ = Poisson's ratio.
υ_c = Poisson's ratio of concrete.
υ_b = Poisson's ratio of rock mass beneath the base of a rock socket.
υ_r = Poisson's ratio of rock mass.
η_c = construction method reduction factor.
ρ = mass density.
ρ_s = ratio of spiral steel reinforcement volume to volume of concrete core.
σ'_1, σ'_3 = major and minor principal effective stresses.
σ'_h = horizontal effective stress.
σ_n = interface normal stress; fluid pressure exerted by concrete at the time of placement.
σ_p' = preconsolidation stress.
σ_v = vertical total stress.
σ_v' = vertical effective stress,
τ = shear strength.
$\tau_{\max}$ = shearing resistance at shaft-rock interface.
θ = shaft rotation.
θ_M = rotation owing to moment.
θ_V = rotation owing to shear.
ϕ = friction angle; resistance factor.
ϕ' = effective stress friction angle.
ϕ_{peak} = peak interface friction angle.
ϕ_{rc} = residual angle of interface friction between rock and concrete.
ϕ_{residual} = residual friction angle.
ψ = angle of dilation.

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APPENDIX A

Survey Respondents

Alabama
Arizona
Arkansas
California
Colorado
Connecticut
Florida
Georgia
Hawaii
Idaho
Illinois
Iowa
Kansas
Kentucky
Massachusetts
Michigan
Minnesota
Missouri
Montana
New Brunswick
New Hampshire
New Jersey
New Mexico
North Carolina
Oregon
Puerto Rico
South Carolina
South Dakota
Tennessee
Texas
Utah
Vermont
Washington

APPENDIX B

Survey Questionnaire and Responses

The survey questionnaire is presented in the following pages. Responses to each question are summarized below each question. Some agencies did not respond to every question.

QUESTIONNAIRE

NCHRP TOPIC 36-12

USE OF ROCK-SOCKETED DRILLED SHAFTS FOR HIGHWAY STRUCTURE FOUNDATIONS

Background and Purpose

Drilled shafts socketed into rock are widely used as highway bridge foundations and can provide high load capacity while controlling displacements when designed and constructed appropriately. However, several challenges for design engineers have been identified in the use of shafts socketed into rock and intermediate geomaterials. These can be grouped into three categories:

- Geotechnical characterization of the rock or intermediate geomaterial
- Analysis and design for axial loading
- Analysis and design for lateral loading.

The purpose of Synthesis Topic 36-12 is to gather information on how these issues have been addressed in the design of highway structures. To accomplish the objective, there will be a literature review, survey of bridge owners from state departments of transportation (DOTs) and toll authorities, and interviews.

This questionnaire is designed to be completed by the state DOT Geotechnical Engineer, assuming that individual has the most knowledge regarding the issues identified above. However, it is recognized that practice varies between states and that other branches within a state DOT may have considerable involvement in drilled shaft engineering. In particular, structural (bridge) engineers responsible for superstructure design may also be involved in foundation design. Therefore, it is recommended that Part V of this questionnaire (Structural Analysis) be completed by the state Bridge Engineer. In addition, it is recommended that Part IV (Design for Axial and Lateral Load) be reviewed by the state Bridge Engineer.

Several questions refer to *intermediate geomaterials* (IGM) as distinguished from *rock*. For purposes of this survey, these two materials are defined as follows:

IGM = cohesive earth material with unconfined compressive strength between 0.5 MPa and 5.0 MPa (5 to 50 tsf) or cohesionless material with SPT *N*-value (N_{60}) greater than 50.

Rock = highly cemented geomaterial with unconfined compressive strength greater than 5.0 MPa (50 tsf).

Part I: Respondent Information

Geotechnical Engineer

Name:

Title:

Agency:

Address:

City: State: Zip:

Phone: Fax: e-mail:

Structural or Bridge Engineer

Name:

Title:

Agency:

Address:

City: State: Zip:

Phone: Fax: e-mail:

Please return the completed questionnaire to:

John P. Turner
Professor, Civil & Architectural Engineering
Department 3295
University of Wyoming
1000 E. University Ave.
Laramie, WY 82071

Phone: 307-766-4265
Fax: 307-766-2221
e-mail: turner@uwyo.edu

After completing the survey, if there are issues pertaining to rock-socketed drilled shafts that you believe are important but that are not addressed adequately by the questionnaire, please feel free to contact the author directly.

Part II: Defining the Use of Rock-Socketed Drilled Shafts by Your Agency

1. On approximately how many projects per year (average) does your agency deal with drilled shaft foundations socketed into rock or IGMs?

☐	None*	(1)	New Brunswick
☐	1–10	(19)	AZ, AL, AR, CT, HI, ID, IL, KY, ME, MI, MN, NH, NJ, NM, SD, TN, UT, VT, WA
☐	10–20	(7)	GA, IA, MO, MT, OR, PR, SC
☐	More than 20 per year		CA, FL, KS, MA, NC, TX

* *If you answered* "None," *skip to Question 5*

2. Please indicate the types of rock or intermediate geomaterials that your agency has dealt with when using rock-sockets. (Check all that apply.)

Igneous rock types		**Sedimentary rock types**		**Metamorphic rock types**	
☐ Granite	13	☐ Conglomerate	13	☐ Slate	8
☐ Rhyolite	5	☐ Sandstone	25	☐ Phyllite	5
☐ Obsidian	0	☐ Mudstone/Shale	24	☐ Schist	12
☐ Diorite	7	☐ Limestone	21	☐ Amphibolite	3

☐	Andesite	7	☐	Dolomite	15	☐	Gneiss	12
☐	Gabbro	5	☐	Chalk	4	☐	Marble	2
☐	Basalt	11	☐	Other (describe)	3	☐	Quartzite	6
☐	Diabase	4				☐	Serpentinite	3
☐	Peridotite	1				☐	Other (describe)	2
☐	Other (describe)	0						

Intermediate Geomaterials:

Alabama	clay-shale
Arkansas	hardclay (8 tsf)
California	mudstone, sandstone, siltstone, phyllite, slate, and weathered rock
Colorado	claystone, siltstone, weakly cemented sandstone
Florida	weathered limestone
Georgia	partially weathered rock
Illinois	weathered limestone, hard clay/shale, cemented sand/sandstone
Iowa	shale, siltstone, sandstone, limestone, dolomite
Kentucky	weathered shale
Massachusetts	till
Michigan	soft shale, hardpan
Minnesota	noncemented sandstone, highly weathered granite
Missouri	softshale
Montana	claystones, siltstones, uncemented sandstones
New Jersey	v. dense sands with $N > 50$
New Hampshire	glacial till
New Mexico	Santa Fe Formation ($N > 75$), of the Rio Grand Rift (indurated, cemented, sands, silts, clay)
North Carolina	weathered rock
Oregon	very soft mudstones, highly weathered sandstones, weakly cemented conglomerates
Texas	clay/shales
Utah:	weak shales and mudstones
Vermont:	glacial till

Washington Has significant deposits of glacial origin. Many have been overridden and overconsolidated by continental glaciation turning them into IGMs by the definition on page 1. A figure, which can be accessed at http://www.dnr.wa.gov/geology/pdf/ri33.pdf contains the unit descriptions. The first four units are encountered in 75% of our shafts.

Other Rock Type Descriptions:

Hawaii:	tuff
Kentucky:	interbedded limestone/shale
Massachusetts:	argillite
Michigan:	iron ore, coal
Minnesota:	meta-graywacke
New Mexico:	gypsum
North Carolina:	partially cemented rock
Oregon:	diatomaceous siltstone

3. Indicate the range of rock-socket diameters and lengths used on your agency's projects.

Range of socket diameters:

Alabama	(3.5–12 ft)
Arizona	(2–6 ft)
Arkansas	(4–9 ft)
California	(2–4 ft)
Connecticut	(2.5–7.5 ft)
Florida	(3–12 ft)
Georgia	(4.5–9.5 ft)
Hawaii	(3–5 ft)
Idaho	(3.5–5 ft)
Illinois	(2–7 ft)
Iowa	(2–10 ft, usually 3.5–4.5 ft)
Kansas	(3–8 ft)
Kentucky	[Typically 4–7 ft (12 ft max.)]
Missouri	(3–10 ft)
Montana	(3.5–10 ft)
New Hampshire	(3–7.5 ft)
New Jersey	(4–6 ft)
New Mexico	(2.5–6 ft)
North Carolina	(3 ft min. to 12 ft max.)
Oregon	(3–8 ft)
Puerto Rico	(3–4.5 ft)
South Carolina	(2–8 ft)
South Dakota	(2.5–10 ft)
Tennessee	(3–8 ft)
Texas	(1.5–8 ft)
Utah	(2.5–4 ft)

Maine	(4.5–7.5 ft)	Vermont	(5–10 ft)
Massachusetts	(Typical 2.5–4 ft; extreme case10 ft)	Washington	(3–10 ft, with understanding that 6 ft and greater may need specialized equipment or methods)
Michigan	(3–5 ft)		
Minnesota	(3–10 ft)		

Range of socket lengths:
Alabama (Geotechnical Foundation Design generally recommends 1 diameter into competent rock. Bridge structures personnel generally drop this tip elevation to 1.5–2 diameters.)

Arizona	(10–30 ft)	Montana	(5–50 ft)
Arkansas	(8–30 ft)	Missouri	(15–120 ft)
California	(15 ft to >300 ft)	New Hampshire	(3–30 ft)
Connecticut	(5–15 ft)	New Jersey	(8–16 ft)
Florida	(3–30 ft)	New Mexico	(6–30 ft)
Georgia	(5–15 ft)	North Carolina	(10–120 ft)
Hawaii	(5–10 ft)	Oregon	(up to 120 ft)
Idaho	(5–12 ft)	Puerto Rico	(10–20 ft)
Illinois	(Typically 5–10 ft, range 3–40 ft)	South Carolina	(2–25 ft)
Iowa	(up to 30 ft)	South Dakota	(30–90 ft)
Kansas	(4.5–20 ft)	Tennessee	(10–30 ft)
Kentucky	(Typically 6–15 ft, approx. 30 ft max.)	Texas	(1–3 shaft diameters)
Maine	(5–23 ft)	Utah	(2–10 ft)
Massachusetts	(Typically 6 ft; several ft extr. case >50 ft)	Vermont	(5–22 ft)
Michigan	(5–17 ft)	Washington	(Usually 2 shaft diameters, so 6–20 ft depending on shaft size)
Minnesota	(5–160 ft)		

4. What group or person in your agency has primary responsibility for design of rock-socketed drilled shafts? (If more than one group within your agency is responsible, please describe briefly the division of tasks below.)

☐ Geotechnical Branch (30)
AL, AZ, CA, CO, CT, GA, HI, ID, IL, IA, KS, KY, ME, MA, MI, MN, MO, MT, NH, NJ, NM, NC, OR, SC, SD, TN, TX, UT, VT, WA

☐ Geology Branch (4)
AZ, CA, KS, MN

☐ Bridge Engineering (structural) (20)
AL, AR, CA, CO, CT, GA, ID, IA, KS, KY, MA, MI, MN, MO, NH, OR, SD, TN, UT, VT

☐ Outside Consultant (11)
CT, FL, HI, ID, IL, IA, KY, MA, NB, NM, PR

Other (explain): Iowa (Input from FHWA)

Division of tasks (if applicable):
AL: Geotechnical is responsible for axial capacity. Bridge is responsible for lateral stability.
CT: All our drilled shaft projects to date have been designed by outside consultants. If we were doing the design in-house, the responsibility would be shared between the geotech and bridge designer.
CO: Geotech provides geotechnical design parameters. Bridge performs design.
FL: Axial—Geotechnical; Lateral—Structural.
GA: Geotechnical—Selection of shafts is recommended foundation type or alternate; bearing pressures, rock-socket length, and tip elevations. Bridge—Selection of shaft diameters, lateral analysis, and possible revision of tip elevations.
KS: Geotechs set base of shaft elevation and recommend side shear and end bearing strengths.

KY: Geotechnical Branch and/or Geotechnical Consultant—Geotech investigation, axial capacities, tip elevations. Division of Bridge Design and/or Structures Consultant—Structural design and detailing, structure plans.
MA: Geotechnical—Dimensions and capacities based on loadings and soil/rock properties. Structural—rebar, concrete, connection designs.
MI: Geotechnical characterizes rock formation and determines rock-socket diameter and length. Bridge Design determines shaft location, shaft loading, and sizes reinforcement.
MN: Geotech determines design bearing capacities and soils and rock properties with consultation with geology. Structures designs final shaft dimensions.
MO: Geotechnical Office provides design criteria and evaluates shaft design based on materials encountered and proposed shaft configuration. Bridge Engineering proposes the layout of foundation units and designs the shaft itself (size, steel configuration, etc.).
NM: Geotechnical Section approves outside consultant designs.
UT: Geotech Branch—rock resistance, L-pile; Structures—structural design.
VT: Geotechnical capacity and lateral analysis is done by the geotechnical branch and structural design is done by the structures group.
WA: Geotechs assess capacity and settlement and provide *p-y* input parameters to bridge office. The structural designer in the bridge office performs the structural design of the shaft assessing shear, moments, and rebar/concrete requirements. They also perform the seismic design using the geotech's *p-y* parameters.

5. In the next 3 years, do you anticipate that the use of rock-socketed drilled shafts by your agency will:

- ☐ Increase (8)
  ID, KS, MA, MO, NB, PR, SC, TN
- ☐ Remain approximately the same (25)
  AL, AZ, AR, CA, CT, FL, GA, HI, IL, IA, KY, ME, MI, MN, MT, NH, NJ, NM, NC, OR, SD, TX, UT, VT, WA
- ☐ Decrease (*none)*

6. Please add any comments you feel would be useful, pertaining to the use of rock-socketed drilled shafts by your agency.

CA: Most result in claims due to the requirement to include "Differing Site Conditions" on all contracts.
IA: Use of drilled shafts has been more frequent in past 2–3 years (above historic use), but may fall off again within next 2–3 years.
KS: Used on high tower lighting and sign structure footings. Used as a contractor's option on some structures. Ease of construction around highway and railway facilities.
MS: Combination of new codes and loadings, issues of scour, and extreme events, are driving the use of deep and/or rock-socketed shafts.
MO: Use is increasing in part due to more consultant bridge design and MoDOT bridge designers gaining more experience in shaft design. Shaft design is cost-competitive with driven pile in many cases and construction in urban areas causes less noise and vibration than driven pile.
New Brunswick: We are beginning to recognize the potential of this type of foundation as an option for bridge foundations in our province. We currently have two projects in the design stage that will use drilled shaft foundations. Where we have limited design experience in-house; most of the questions in the survey are left unanswered. We look forward to reviewing the results as a way to see how other agencies approach these designs, as we move toward the consideration of drilled shafts as an option in the future.
NH: Emphasize that the design of drilled shafts should include consideration of the drilled shaft construction methods and constructability issues.
OR: Most of our shafts are socketed into bedrock, either with or without end bearing resistance. Many times we need rock embedment to resist high lateral loads associated with high seismic loading conditions, sometimes coupled with soil liquefaction.
SC: Finding some way to equate different rock drilling rigs/equipment capabilities to varying rock strengths.
SD: 99% of the drilled shafts done in this state are done in shale bedrock.

WA: When we want to have a rock socket of a certain length and recognize that the rock may be variable in elevation we include the following provision in our shaft special provision. With this special in place, the contractor can tie the reinforcing cage prior to excavating, excavate to rock, construct the rock socket, and trim the cage to fit. Excavation to tip elevation, cage placement, and the concrete pour can be complete in one shift this way. We pay for the steel that is cut off from the bottom of the cage, but feel that it is well worth the investment by lowering our risk of a blow-in or caving as the shaft does not have to sit open for days while the cage is tied. When the contract requires a minimum penetration into a bearing layer, as opposed to a specified shaft tip elevation, and the bearing layer elevation at each shaft cannot be accurately determined, add subsection 3.05.E as follows: For those shafts with a specified minimum penetration into the bearing layer and no specified tip elevation the Contractor shall furnish each shaft steel reinforcing bar cage, including access tubes for cross-hole sonic log testing in accordance with subsection 3.06 of this Special Provision, 20% longer than specified in the plans. The Contractor shall add the increased length to the bottom of the cage. The contractor shall trim the shaft steel reinforcing bar cage to the proper length prior to placing it into the excavation. If trimming the cage is required and access tubes for cross-hole sonic log testing are attached to the cage, the Contractor shall either shift the access tubes up the cage or cut the access tubes provided that the cut tube ends are adapted to receive the watertight cap as specified.

Part III: Characterization of Rock or Intermediate Geomaterial (IGM)

7. Check the methods used by your agency to determine depth to bedrock for the purpose of drilled shaft foundation engineering.

☐ Standard Penetration Test (SPT) refusal (22) AR, CA, CO, FL, GA, HI, IL, IA, KS, MA, MI, MN, MO, MT, NJ, NM, NC, OR, PR, SC, UT, VT

☐ Cone Penetration Test (CPT) refusal: (3) KS, MN, MO

☐ Coring and inspection of core samples (30) AL, AZ, AR, CA, CO, CT, FL, GA, HA, ID, IL, IA, KS, KY, ME, MA, MI, MN, MO, NH, NJ, NM, NC, OR, PR, SC, TN, UT, VT, WA

☐ Geophysical methods (specify)
AZ, CA, ID (seismic refraction), KS, KY (currently using resistivity and microgravity on a project where we are considering drilled shaft foundations. We have not previously used geophysical methods for drilled shaft investigations), MT, UT, (seismic refraction), WA

☐ Other (specify—provide details if possible)
CA (2.25 in diameter terry cone driven to refusal. Please note that no one method is solely relied upon), IL, IA (laboratory UC or other tests), MN (pressuremeter), NJ (point load strength test), NM (RQD/RMR/joint orientation/water/polymer slurry only), SD (California retractable plug sampler to extract samples), TX (Texas Cone Penetrometer—for information see the following websites: http://txdot-manuals/dynaweb/colbridg/geo—go to Chapter 2, Section 4, Field Testing for Design Charts—go to Chapter 4, Design Guidelines: http://txdot-manuals/dynaweb/colmates/soi/@Generic__BookView;cs=default;ts=default; go to Section 32 Tex—132-E Texas Cone Penetration Test).

8. How does your agency distinguish between rock, soil, and intermediate geomaterials?

☐ Defined in the same way as stated on page 1 of this questionnaire (24) AL, AR, CA, CO, FL, HI, ID, IL, KS, KY, ME, MI, MN, MO, MT, NM, OR, PR, SC, SD, TN, UT, VT, WA

☐ Other: summarize below
AZ: Typically, we classify as either soil or rock only by the use of test borings.

CT: We generally do not try to quantify IGMs. We may have some glacial tills/weathered bedrocks overlying a hard bedrock that would be an IGM, but we do not usually spend much time defining its' engineering properties for the design of drilled shafts.
CO: Some very weathered claystone is classified as rock even if it is weaker than IGM, as described in the background and purpose section above.
GA: Soil-drilled and sampled with earth augers, SPT < 50±, drilled shaft bearing pressure < 30–40 ksf; IGM-drilled with earth and/or rock augers, SPT > 50±, drilled shaft bearing pressure > 40 ksf, < 75 ksf; rock material below auger refusal sampled with diamond core drilling, drilled shaft bearing pressure > 75 ksf.
IL: Experience combined with field observation of drilling operation (difficulties, change of drilling tools, etc.)
IA: Classify as IGM? Rock if of "sedimentary rock" geologic origin. Classify as soil if of glacial, alluvial, similar deposition.
KY: We have very few IGMs and if we have them, they are typically a weathered zone of shale in a transition from residual soil to interbedded limestone and shale. This material is typically neglected for drilled shaft design.
MA: We have a clear distinction between rocks and soils, based on coring use.
ME:
NH: For classification purposes on test boring logs, differentiation of bedrock vs. IGM or soil based on geologic interpretation of boring samples. For drilled shaft analysis, would generally use the definitions on page 1 (e.g., weathered bedrock would be classified on the boring log as bedrock, but would be analyzed as an IGM).
NJ: Based on the coring results, RQD and recovery, and engineering judgment; e.g., RQD < 30% may be considered as IGM not sound rock.
NC: Definition of Rock—SPT and refusal—"Rock" is defined as a continuous intact natural material in which the penetration rate with a rock auger is less than 2 in. (50 mm) per 5 min of drilling at full crowd force. This definition excludes discontinuous loose natural materials such as boulders and man-made materials such as concrete, steel, timber, etc.
TX: Our design methodology does not require specific designation of rock, soil, or IGM. Design is generally based on the strength testing, regardless of material designation.

The following is a list of rock properties that may be required or recommended to apply design methods specified in the FHWA *Drilled Shaft Manual*, as well as for other published design methods used for rock-socketed drilled shafts:

q_u	=	unconfined compressive strength (units of F/L^2)
RQD	=	Rock Quality Designation
ϕ_{RC}	=	effective stress angle of friction between the rock or IGM and concrete
E_{core}	=	Young's modulus of rock or IGM core (units of F/L^2)

Rock Mass Quality as defined in terms of:

RMR	=	Rock Mass Rating (Bieniawski 1974)
Q	=	Norwegian Geotechnical Institute rating (Barton 1974)
c'_i and ϕ'_i	=	instantaneous values of cohesion and friction angle for Hoek–Brown nonlinear strength criteria of fractured rock masses

9. For each rock property, check the appropriate box indicating whether your agency determines this property for rock-socket design and, if so, the method used to determine the property:

q_u ☐ Always ☐ Never ☐ Varies

Always: (23) AL, AZ, CT, FL, HI, ID, IL, KS, ME, MA, MI, MN, MO, MT, NJ, NM, OR, SC, SD, TN, UT, VT, WA
Never: (0)
Varies: (10) AR, CA, CO, GA, IA, KY, NH, NC, PR, TX

Method:

ASTM D2938 or AASHTO T-226: Uniaxial Compressive Strength (14)
AL, AZ, IA, KY, MN, MO, NH, NM, OR, SC, TN, TX, UT, VT
Point Load Tests and/or Uniaxial Compression of intact core: (3)
MA, MI, WA
Maine (ASTM D7012-04) ??

RQD ☐ Always ☐ Never ☐ Varies

Always: (28) AL, AZ, AR, CA, CO, CT, GA, HI, ID, IL, IA, KS, ME, MA, MI, MN, MO, MT, NH, NJ, NC, OR, PR, SC, TN, UT, VT, WA
Never: (1) SD
Varies: (3) FL, KY, TX

ϕ_{RC} ☐ Always ☐ Never ☐ Varies

Always: (2) KS, MT
Never: (23) AL, AR, CT, FL, GA, HI, ID, IL, KY, ME, MA, MI, MN, NH, NM, NC, OR, PR, SC, SD, TX, UT, VT
Varies: (7) AZ, CA, CO, IA, NJ, TN, WA

Method: AL (Relies on charts), AZ (Estimate from AASHTO Manual), IA (theoretical),
WA (Usually use published textbook values based on rock type)

E_{core} ☐ Always ☐ Never ☐ Varies

Always: (5) KS, ME, MN, UT, VT

Never: (11) AL, AR, CA, HI, ID, MT, NM, PR, SC, SD, TX

Varies: (16) AZ, CO, CT, FL, GA, IL, IA, KY, MA, MI, NH, NJ, NC, OR, TN, WA

Method:

AL: Correlation charts between q_u and E
IA: Theoretical
KY: Correlation with UC strength
ME: ASTM 7012-04
MA: Goodman, Jack, and tables/charts
MI: Calculated from Ultrasonic Velocity test (ASTM D2845) or approximated from figures and tables in section 4 of the AASHTO Standard Specs
MN: ASTM D3148
NH: E_{intact} determined from q_u test, then correlated to $E_{in\ situ}$ through RMR or other methods
OR: From ASTM D2938 results with measured strains
UT: either unconfined compression test or from AASHTO table
VT: ASTM D3148
WA: Usually use published textbook values based on rock type

RMR ☐ Always ☐ Never ☐ Varies

Always: (5) MA, NH, NM, TN, UT

Never: (15) AL, AR, FL, HI, IL, KY, ME, MI, MN, MT, NJ, PR, SC, SD, TX

Varies: (12) AZ, CA, CO, CT, GA, ID, IA, KS, NC, OR, VT, WA

Q ☐ Always ☐ Never ☐ Varies

Always: *none*

Never: (29) AL, AZ, AR, CA, CO, CT, FL, GA, HI, ID, IL, IA, KY, ME, MA, MI, MN, MT, NH, NJ, NM, OR, PR, SC, SD, TX, UT, VT, WA

Varies: (3) KS, NC, TN

c'_i and ϕ'_i ☐ Always ☐ Never ☐ Varies

Always: *none*

Never: (24) AL, AR, CT, FL, GA, HI, ID, IL, IA, KS, KY, ME, MA, MI, MN, MT, PR, SC, SD, TN, TX, UT, VT, WA

Varies: (8) AZ, CA, CO, NH, NJ, NM, NC, OR

10. List below any in situ test methods that are used by your agency to correlate with rock or IGM properties *or* to correlate directly to rock-socket design parameters (e.g., side or end bearing resistance).

In Situ Test	State	Property or Design Parameter
Standard Penetration Test (SPT)	CA	not stated
	HI	strength
	IL	inch penetration per 100 blows (no property stated)
	MO	correlation to q_u
	NH	side and end bearing resistances
	WA	friction angle for IGMs
	FL	strength
Pressuremeter Test (PMT)	AZ	shear values
	CA	rock mass modulus
	MN	stiffness (rock mass modulus)
	OR	correlation with *p-y* curves
Borehole (Goodman) Jack	MA	rock mass modulus
Dilatometer	CA	rock mass modulus
TxDOT Cone Penetrometer	TX	correlate to side and tip resistances

11. Indicate by marking the boxes whether your agency uses any of the following tools for evaluating characteristics of rock below base elevation:

☐ Coring into the rock below the bottom of the shaft *after* the excavation to base elevation is complete; if so, to what depth?

AZ: 3B or minimum of 10 ft
AL: typically 10 ft unless specified otherwise
FL: ≥10 ft
GA: 6 ft
MO: 10 ft below the bottom of the shaft for end bearing design; not required when designed for side friction only
MT: 50 ft
NJ: not stated
NM: 3 diameters
NC: 5 ft
TX: at least 5 ft deep or a depth equal to the shaft diameter, whichever is greater.

☐ Coring into rock below bottom elevation *prior* to excavating the shaft.

(27) AL, AZ, AR, CA, CT, FL, GA, HI, ID, IL, IA, KS, KY, MA, MI, MN, MO, NH, NJ, NM, NC, OR, PR, TN, UT, VT, WA

☐ Inspection of core holes at the bottom of the socket using "feeler rods."

(5) AL, GA, MA, NC, TN

☐ Inspection of core holes using fiberoptic cameras.

(5) AZ, NJ, NM, NC, UT

Other: CA: Visually inspect drilled hole and cores and/or cuttings that are removed.
IL: Visual inspection and classification of rock core by an experienced geologist.
ME: Camera inspection of rock-socket base and extending borings during design stage to depth below expected bottom of rock socket.
NC: 10 lb weight, SID camera, or use temporary casing to inspect the base by the engineer or the contractor.
SC: Corings into rock below shaft bottom during design represent expected rock below base.
UT: Visual inspection; many times, the rebar cage is designed to go to the bottom of the boring (in shorter shafts)—this verifies depth.

12. If your agency has experience in the design and construction of rock sockets with any of the materials listed below, please check the appropriate box and provide information on test methods (field or laboratory) that you have used to characterize the material properties

☐ Weak lime rock (11) AZ, FL, GA, IA, KS, MN, MO, NC, SC, TX, UT

State: Property; Test Method; Correlations Used
AZ: This applies to all rock types listed below: shear and end bearing; unconfined compressive strengths and RQD; AASHTO Guidelines
FL: Coring, q_u, q_t, RQD, Recovery (%), SPT
GA: Coring, RQD, compressive tests, split tensile
IA: Strength, skin friction/end bearing; lab UC on cores, O-cell tests
KS: Core, RQD RMR q_u
MN: Strength and stiffness; unconfined compression
MO: Compressive strength; q_u on core sample
NC: Typically we core the rock and perform unconfined compression tests
SC: Unconfined; load test
TX: q_u, skin friction, point bearing; ASTM, Tex-132-E; TxDOT *Geotechnical Manual* correlations
UT: Same mentioned in Question 9

☐ Soft shales or marls (14) AL, AZ, CA, GA, IA, KS, KY, MI, MO, NM, SC, SD, TX, UT

State: Property; Test Method; Correlations Used
AL: Rock strength, thickness and spacing of discontinuities; unconfined compression testing where possible, logged by a professional geologist from the cored rock (for all rock types checked)
CA: Unconfined strength; triaxial test
GA: Coring, RQD, compression tests
IA: Strength skin friction/end bearing; lab UC on cores and O-cell tests
KS: As above
KY: Unconfined compression, slake durability index
MI: Unconfined compressive strength; ASTM D2166
MO: Compressive strength; q_u on core and correlations with SPT; Texas DOT correlations modified by

MoDOT
NM: q_u/triax shear; AASHTO T296; Alpha method
SC: Triaxial; load tests
SD: Soil strengths; unconfined compression test; skin resistance compared to pull test on steel pk rod
TX: q_u, skin friction, point bearing; ASTM, Tex-118-E, Tex-132-E; TxDOT *Geotechnical Manual* correlations
UT: Same as mentioned in Question 9

☐ Weathered and highly fractured rock (20)
AL, AZ, CA, GA, HI, IA, KS, KY, MA, MI, MN, MO, NM, NC, OR, SC, SD, TX, UT, WA

State: Property; Test Method; Correlations Used
AR: Visual observations of rock condition
GA: Coring, RQD, compression tests
HI: Strength; unconfined compression
IA: Strength skin friction/end bearing; lab UC on cores and O-cell tests
KS: As above
KY: We may use Slake Durability Index in shale and sometimes sandstone
MA: RMR/q_u
MI: Unconfined compressive strength; Point Load Test ASTM D5731; correlations included in test procedure
MN: Strength and stiffness; SPT or pressuremeter
MO: RQD
NM: phi'; N_{60}; Mayne and Harris
OR: Shear strength; SPT, judgment based on experience, often treated as very dense granular soil; Meyerhof or Peck, Hanson, Thornburg
SC: SPT
SD: Soil strengths; unconfined compression test; skin resistance compared to pull test on steel pk rod
TX: q_u, skin friction, point bearing; ASTM, Tex-132-E; TxDOT Geotechnical Manual correlations
UT: Same mentioned in Question 9
WA: RQD and unconfined compressive strength; drilling and Point Load

☐ Karst (9) AL, AZ, FL, GA, KS, KY, MI, NM, TX

State: Property; Test Method
AZ: Core into rock after the excavation to check for voids
FL: Coring, q_u, q_t, SPT
GA: Coring, RQD, compressive tests, split tensile
KS: Seismic
KY: Rock Core Recovery
NM: Discontinuities; test pits/seismic shear wave
TX: q_u, skin friction, point bearing; TxDOT Geotechnical Manual correlations

☐ Rock with steeply dipping discontinuities (7) AL, AR AZ, CA, GA, NM, WA

State: Property; Test Method; Correlations
AR: Visual observations of rock condition
AZ: Down-the-hole camera to check for poorly oriented joint sets
GA: Coring, RQD, compressive tests
NM: Modulus; RMR
WA: RQD and unconfined compressive strength; drilling and Point Load

☐ Interbedded rock with alternating strong and weak strata (11)
AL, AR, AZ, CA, GA, IA, KS, KY, MO, NM, TX

State: Property; Test Method

AR: Visual observations of rock condition
GA: Coring, RQD, compressive tests
IA: Strength skin friction/end bearing; lab UC on cores, O-cell tests
KS: Core, q_u, RMR, RQD
KY: Unconfined compression, Slake Durability Index
MO: Compressive strength; q_u from representative core samples
NM: Side shear/modulus; Rowe and Armitage
TX: q_u, skin friction, point bearing; ASTM, Tex-118-E, Tex-132-E

☐ Hard, intact rock (22) AL, AZ, CA, CT, GA, HI, ID, IA, KY, ME, MA, MI, MN, MO, NH, NM, NC, OR, SC, TX, VT, WA

State: Property; Test Method; Correlations
AL:
AZ:
CA:
CT: Unconfined compressive strength
GA: Coring, RQD, compressive tests
HI: Strength; unconfined compression
ID: Unconfined compressive strength; ASTM D2938
IA: Strength skin friction/end bearing; Lab UC on cores O-cell tests
KY: Unconfined compression
ME: q_u and E; D 7012-04
MA: q_u; point load: Ip; 25 Ip $= q_u$
MI: Unconfined compressive strength; ASTM-C42
MN: Strength and stiffness; unconfined compression test
MO: Compressive strength; *qu* from core samples
NH: Intact compressive strength; unconfined compression test
NM: q_u; RMR
NC:
OR: Unconfined compressive strength; ASTM D2938
SC: Unconfined; FHWA methodology
TX: *qu*, skin friction, point bearing; ASTM, Tex-132-E; TxDOT *Geotechnical Manual* correlations
VT: *qu*; ASTM D2938; FHWA IF-99-025
WA: RQD and unconfined compressive strength; drilling and point load

13. Identify any other issues pertaining to IGM or rock characterization that you think should be addressed by the Synthesis.

MO: Limited Osterberg load cell testing has indicated that we significantly overdesign shafts in IGMs based on compressive strength values from q_u testing. We need low cost in situ or other test methods for obtaining ultimate capacities in IGMs.
NC: NCDOT and the NC State University conducted research to determine *p-y* curves for soft weathered rock loaded horizontally.
OR: In Question 9, is anyone actually measuring or estimating the "s" and "m" dimensions of the rock mass for the Carter and Kulhawy equation? Also, is anyone estimating borehole "roughness" and using the Horvath (1983) equation? We are not because we have no real way of knowing if this can be accomplished in the field.
UT: How are states handling discontinuities in design; how are strength values of the discontinuities being determined, etc.?
WA: In Washington State, our shaft lengths are rarely designed to carry the applied axial loads. Most shafts have very significant lateral capacity demand owing to earthquake loading. Tip elevations are often set to meet lateral capacity requirements. Very little information is available on the lateral capacity or lateral behavior of shafts in IGMs subjected to lateral loads. The effects of group loading in IGMs are also not well-documented.

Part IV: Design for Axial and Lateral Load

***Note*:** The terms "base resistance," "tip resistance," and "end-bearing resistance" are used by various agencies; all refer to the resistance developed beneath the tip of a deep foundation.

14. When designing for axial load of rock-socketed shafts, does your agency account for:

- ☐ Both side and base resistances (25) AL, AZ, CA, CO, CT, FL, GA, IA, KS, KY, ME, MI, MN, MO, MT, NM, NC, OR, PR, SC, TN, TX, UT, VT, WA
- ☐ Side resistance only (10) CA, HI, ID, MA, MN, MO, NH, NJ, SD, TN
- ☐ Base resistance only (7) AR, CA, ID, ME, MA, MO, TN

Comments:

AZ: Rely mainly on side resistance with reduced end bearing.
CA: Depends on anticipated methods of construction.
IL: Depends on the elastic deformation. Generally, not both side and end.
MO: Evaluated on a case-by-case basis. Typically end bearing only in hard rock and side resistance only in alternating hard and soft rock layers.
NH: Rock-socket lengths typically controlled by lateral load with sufficient geotechnical capacity provided by side shear only. Would consider using a portion of the end bearing geotechnical capacity in combination with full side shear, if needed, to avoid extending socket length beyond what may be needed for lateral loads.
NC: Depends on our design; we might use base or side resistance but most of the time we use both.
OR: Combine side and base resistance only in very ductile rock formations as described in the FHWA *Drilled Shaft Design Manual.*
TN: Geotech Section provides parameters for both. Structure designer decides which to use.
UT: In wet conditions we will, many times, discount base resistance; with the current LRFD code we have been using either side resistance only (most of the time) or base resistance, based on deflection.

15. For calculating side resistance of rock sockets, please indicate the reference(s) associated with the method(s) used by your agency (mark all that apply):

- ☐ O'Neill and Reese (1999) Publication No. FHWA-IF-99-025, *Drilled Shafts: Construction Procedures and Design Methods*
  (26) AL, AZ, CA, CT, GA, HI, ID, IA, KS, KY, MA, MI, MN, MO, MT, NH, NJ, NM, NC, OR, PR, SC, TN, UT, VT, WA
- ☐ Horvath and Kenney (1979)
  (8) CA, HI, ID, IL, ME, OR, SC, TN
- ☐ Rowe and Armitage (1984)
  (1) NM
- ☐ Carter and Kulhawy (1988)
  (6) CA, IL, ME, NC, SC, WA
- ☐ Other (please cite reference or provide a brief description)
  (6) AZ: (AASHTO 2002)
  KS: (Results from O-cell tests)
  FL: McVay et al. (1992)
  NH: (2002 AASHTO bridge code)

OR (not stated)
TX (TxDOT Design Method—See Chap. 4 of the *Geotechnical Design Manual* (website)

16. What, if any, computer programs are used by your agency for analysis of rock-socket response to axial loading?

☐ SHAFT (14) AL, AZ, CA, GA, KS, MN, MT, NJ, NM, NC, OR, PR, SC, UT
☐ FBPIER (6) CT, FL, MN, NC, TN, VT
☐ ROCKET (0)
☐ CUFAD (0)
☐ Other (name of program) (4)
FL: FB Deep
IL: In-house spreadsheet based on Pells and Turner
KY: In-house spreadsheets
TX: WinCore–TxDOT program for the design of drilled shafts

17. Specify the range of values used by your agency for either Factor of Safety (FOS) or Resistance Factor (ϕ_s) applied to rock-socket ultimate side resistance in design (if applicable, specify by rock type).

AL: FOS 3.0
AZ: All, FOS 2.5
CA: Hard rock, FOS 2.0–2.5 or ϕ_s 1 or 0.75
CT: AASHTO ASD, LFD, or LRFD recommended values regardless of rock type
FL: By AASHTO LRFD
GA: Weak IGM or hard granite FOS 2.5
HI: Tuff, ϕ_s 0.65
ID: Igneous (basalt), ϕ_s 0.55–0.65
IL: All, FOS 2.5
IA: IGM and "Rock," FOS 2–2.5
KY: All, FOS 2 (if load tested) to 3
MA: All, FOS 2–2.5 or ϕ_s 0.55–0.65
ME: Schist, FOS 2.5
MI: All, FOS 3
NH: All, FOS 2.5
NJ: All rocks, FOS 2
NM: All, FOS 2.0
NC: All, FOS 2.5–3.00, We reduce the FOS if we perform a load test
OR: All, FOS 2.5
SC: All, FOS 2–3, ϕ_s 0.4–0.7
SD: Shale, FOS 2.0?
TX: All, FOS 3
UT: All types, ϕ_s 0.55
VT: All, FOS 2.5
WA: All, FOS 3.0 Static, 1.65 Seismic

18. For calculating base resistance of rock sockets, please indicate the reference(s) associated with the method(s) used by your agency (mark all that apply):

☐ O'Neill and Reese (1999) Publication No. FHWA-IF-99-025, *Drilled Shafts: Construction Procedures and Design Methods*
(24) AL, AR, CA, CT, GA, ID, IA, KS, KY, ME, MA, MI, MN, MO, MT, NH, NJ, NM, NC, OR, PR, UT, VT, WA

☐ *Canadian Foundation Engineering Manual* (1979)
(3) CA, IL, ME

☐ Zhang and Einstein (1998)
(2) ID, MA

☐ Carter and Kulhawy (1988)
(3) AR, CA, WA

☐ Other (please cite reference or provide a brief description)
AZ: AASHTO 2002
KS: O-cell tests results
KY: Experience and judgment

NH: 2002 AASHTO Bridge code
NM: Rowe and Armitage
TN: Use Soils and Geology Allowables and Section 4 of AASHTO Specs.
TX: TxDOT Design Method—See Chap. 4 of the *Geotechnical Design Manual* (website)
WA: AASHTO LRFD Manual

19. Specify the range of values used by your agency for either Factor of Safety (FOS) or Resistance Factor (ϕ_s) applied to rock-socket ultimate base resistance in design:

AL: FOS 3.0
AR: Sandstone or shale, FOS 2.5
AZ: All, FOS 2.5
CA: All, FOS 2.0 or ϕ_s 1 or 0.75
CT: AASHTO ASD, LFD, or LRFD recommended values regardless of rock type
FL: Side/end: 0.55; side only: 0.60
GA: Weak IGM or hard granite, FOS 2.5
ID: Igneous (basalt), ϕ_s 0.5
IL: All, FOS 2.5
IA: IGM and "rock," FOS 2–2.5
KY: All, FOS 2 (if load tested) to 3
MA: All, FOS 2–2.5 or ϕ_s 0.5
ME: Schist, FOS 2.5
MI: All, FOS 3
MN: FOS 2.5
NH: All, FOS 2.5
NJ: All rocks, FOS 2
NM: All, FOS 2.0
NC: All, FOS 2.5–3.0
OR: All, FOS 2.5
SC: All, FOS 2–3, ϕ_s 0.4–0.7
SD: Shale, FOS 2.0?
TN: All, FOS 2.5
TX: All, FOS 2
UT: All types, ϕ_s 0.5
VT: All, FOS 2.5
WA: All, FOS 3.0 Static, 1.65 Seismic

20. If you include both side and base resistances in design of rock sockets, explain briefly how you account for the relative contribution of each to the socket axial resistance

CA: Must determine the amount of each that can be mobilized at our allowable movement at the top of the pile.
CT: The relative contributions would be based on the computed displacement/strain of the drilled shaft. If load test data were available, the strain compatibility would be validated or refined based on the actual test data.
FL: Based on compatibility.
IL: For weak IGM.
IA: Both are typically limited by settlement criteria for both allowable and ultimate loads.
KS: In good hard rock most of the load is stripped off in side shear. In shales, we assume that side shear and end bearing act together; either O-cell testing at site or extrapolation of previous testing.
KY: Evaluate strain compatibility if O-cell load test is run. If no load test, use higher FS (3).
ME: Two projects were designed in accordance with AASHTO 4.6.5.3, assuming that axial loads are carried solely by end resistance, as the strains required for full mobilization of both end and side resistance is incompatible. This design approach was later altered on one project to assume conservative, simultaneous mobilization of both end and side frictional resistance. That design approach ignored side resistance in the upper 5 to 15 ft may (based on a minimum required value of RQD and q_u, determined by the geotechnical engineer). For the remainder of the side walls, partial contribution is assumed, in addition to partial mobilization of full end bearing.
MI: Seek to design socket to have side friction capacity 2.0 to 2.5 times applied load. When end bearing contribution is added, seek to show FS greater than or equal to 3.0.
NM: Osterberg and Gill (length/modulus ratio).
NC: This assumption will depend on engineering judgment and the method of construction.
OR: According to methods described in FHWA manual; determine the resistance available from the side and base independently based on a given relative shaft settlement and then add them together.
SC: Assume side fully mobilized and 5% diameter settlement not necessary to mobilize end resistance in rock.
TN: Geotech Section opinion is that with the rock type and strength we have and using a safety factor of 2.5, a relatively small mobilization of side and end bearing occurs; therefore, it is okay to use a combination of both. Structures Designer typically uses just one or the other.

TX: TxDOT Design Method—See Chap. 4 of the *Geotechnical Design Manual* (website cited above).
WA: See the attached pdf discussing WSDOT procedures for designing drilled shafts in rock and IGMs.

21. When a bridge is supported on a shallow footing that is supported on a rock-socketed drilled shaft (as opposed to a mono shaft), does your design procedure account for the contribution of the footing to the foundation capacity?

☐ Yes (*none*)

☐ No (25) AL, AR, CA, CT, FL, GA, HI, IL, IA, KS, KY, MA, MI, MN, MT, NH, NM, NC, SC, SD, TN, TX, UT, VT, WA

☐ Not applicable (6) AZ, ID, ME, NJ, OR, PR

If you answered "Yes," please provide a brief description of your analysis to account for the footing contribution: (*none*)

22. For analysis of rock-socketed shafts under lateral loading, please indicate the methods and/or references associated with methods used by your agency (mark all that apply).

☐ Equivalent Cantilever Method (Davisson 1970) (5) KS, MA, NH, NC, SC

☐ Broms Method (Broms 1964) (5) KY, MT, OR, SC, TN

☐ *p-y* method of analysis (26) AZ, AR, CA, CT, FL, GA, HI, IL, IA, KS, KY, MA, MI, MN, NH, NJ, NM, NC, OR, PR, SC, TN, TX, UT, VT, WA

☐ Characteristic Load Method (Duncan et al. 1994) (*none*)

☐ Zhang, Ernst, and Einstein (2000) "Nonlinear Analysis of Laterally Loaded Rock-Socketed Shafts" (1) MA

☐ Reese, L.C. (1997) "Analysis of Laterally Loaded Piles in Weak Rock" (8) GA, ID, MI, MT, NJ, NC, OR, TX

☐ Carter and Kulhawy (1992) "Analysis of Laterally Loaded Shafts in Rock" (1) NJ

☐ Other (please cite reference or provide a brief description)
ME: FB Pier evaluation
SC: Some lateral load resisting
WA: S-Shaft Program developed by M. Ashour and G. Norris of UNR along with J.P. Singh of J.P. Singh & Associates. Model is based on strain wedge theory

23. What, if any, computer programs are used by your agency for analysis of rock-socket response to lateral loading?

☐ LPILE^{PLUS} (23) AL, AZ, CA, GA, HI, ID, IA, KY, MA, MI, MN, MT, NH, NJ, NM, NC, OR, PR, SC, TX, UT, VT, WA

☐ COM624P (17) AR, CA, CT, GA, ID, IL, IA, KS, KY, ME, MA, NJ, NC, OR, PR, TX, VT

☐ FBPIER (8) FL, MI, MN, NJ, NC, TN, TX, VT

☐ Finite-Element Method (specify program)
NC: Flac

☐ Other (provide name of program)
NH: Group 6.0
WA: S-Shaft Program developed by M. Ashour and G. Norris of UNR along with J.P. Singh of J.P. Singh & Associates. Model is based on strain wedge theory

24. If you use the *p-y* method of analysis, describe briefly how you determine the *p-y* relationships for rock

☐ Published correlations between rock properties and *p-y* curve parameters
(4) CA, KY, NJ, VT

Reference(s): CA: LPILE Manual

☐ Correlations built into computer code (specify program)
(22) AZ, AR, CA, CT, FL, GA, HI, IL, IA, MA, MI, MN, NH, NM, NC, OR, PR, SC, TN, TX, UT, VT (*all of the above states use LPILE*)

☐ In-house correlations based on agency experience
(2) NC, OR

☐ Educated guess
(2) MA, OR

☐ Other (describe)
CA: Pressuremeter Testing
MN: in situ test
NC: Research
OR: Limited pressuremeter data in soft rocks
WA: Reese *p-y* curves for vuggy limestone are derived using elastic theories. For basalt, using engineering judgment, we typically define *y* as 0.01B, assuming 0.5% strain is the typical range over which basalt behaves linearly and that is the rock within 2B that resists the load. Therefore, $y = 0.005(2B)$ or 0.01B. We then use correlations or published values to determine Young's modulus *E*. Typically, this is about 10,000 ksi for basalt. We then use the unconfined compressive strength from point load tests along with *E* to define the curve. For example, if q_u is 22.8 ksi we would take the 10,000 ksi (*E*) value and divide by the q_u to get about 440. The *p-y* curve would then be defined by a straight line beginning at the origin with a slope of 440 q_u. In highly fractured rock, the engineer would use judgment to change strain that defines *y*1, thus flattening the *p-y* curve.

25. On projects completed by your agency, which of the following design considerations control rock-socket length (approximately)?

☐ Axial capacity

AL	Not really	KS	50%	NJ	65%
AZ	99%	KY	30%	NM	80%
CA	70%	ME	100%	NC	30%
CT	50%	MA	30%	OR	35%
HI	100%	MI	50%	SC	40%
ID	100%	MN	90%	SD	100%
IL	50%	MT	50%	TX	95%
FL	70%			UT	80%

☐ Lateral load response

AL:		KY:	60%	NC:	70%

AZ:	1%	MA:	60%	OR:	60%
AR:	100%	MI:	50%	SC:	60%
CA:	20%	MN:	10%	TN:	
CT:	50%	MT:	50%	TX:	5%
GA:	10%	NH:	90%	UT:	20%
IL:	40%	NJ:	35%	VT:	100%
KS:	50%	NM:	10%	WA:	100%
FL:	30%				

☐ Construction-related

CA:	10%	IL:	10%	MA:	10%
GA:	90%	KY:	10%		

☐ Sharing of load between drilled shaft and footing
Puerto Rico: 100%

☐ Other (explain)
IA: No information available
KS: Minimum of 1.5 x shaft diameter
NM: 10% scour
OR: 5% scour
TN: 1.5 x socket diameter

26. Please identify any other issues pertaining to rock-socket analysis/design that you feel should be addressed in this synthesis.

FL: In karst, check for voids below tip.
IL: Bureau of Bridges and Structures, and consultants.
KS: Pertaining to Question 17, at the LFD load, the settlement should be less than some acceptable value. We are using an arbitrary value of approximately ¼ in. This will vary depending on the bridge type and span length.
MA: The design/analysis of highway structures foundation (traffic signals, etc.). Construction practices and QA/QC and their influences on design assumptions.
NH: Provide additional guidance for using side shear and end bearing in combination and provide simplification of side resistance equations for cohesive soils contained in FHWA-IF-99-025.
OR: What agencies are using the AASHTO methods for drilled shaft design in rock?
UT: Concerns with appropriate lateral analysis methods; that is, is LPILE appropriate to be using with rock sockets?

Part V: Structural Analysis

27. What branch or group within your agency is responsible for structural design of rock-socketed drilled shafts?

AL: Bridge Bureau
AZ: Bridge Group
AR: Bridge Design
CA: Division of Engineering Services/ Structure Design
CT: No drilled shaft design has been done with in-house engineering staff
GA: Office of Bridge Design
HI: Bridge Design Section or Structural Consultants
ID: Bridge Design
ME: Bridge Program
MN: Bridge Office
MT: Bridge
NH: Bridge and Geotechnical Sections
NJ: Structural and Geotechnical Engineering Units
NM: Bridge Section
OR: Technical Services, Region Technical Centers
PR: None, done by consultants
SC: Bridge Design Section
SD: Bridge Design

IA: Office of Bridges and Structures
KS: Bridge Design Section
KY: Division of Bridge Design
MA: Bridge Designer (either in-house or consultant) after consultation with Geotechnical Section
TN: Division Structures
TX: Geotechnical Branch
UT: Structures Division
WA: Bridge and Structures Office
FL: Geotech for resistance; Structures for structure design

28. Mark all of the applicable references/codes used by your agency in the structural design of rock-socketed drilled shafts:

☐ O'Neill and Reese (1999) Publication No. FHWA-IF-99-025, *Drilled Shafts: Construction Procedures and Design Methods*
(15) AR, CA, CT, FL, ID, IA, KS, KY, MA, NH, NJ, NC, OR, PR, VT

☐ ACI 318, *Building Code Requirements for Structural Concrete*
(3) IL, NJ, WA

☐ AASHTO, *Bridge Design Specifications*
(26) AZ, AR, CA, CT, FL, GA, HI, ID, IL, IA, KS, KY, MA, MN, NH, NJ, NM, NC, OR, SC, SD, TN, TX, UT, VT, WA

☐ ACI 336, *Design and Construction of Drilled Piers*
(2) NJ, WA

29. For structural design of drilled shafts, does your agency currently use Load Factor Design (LFD), Load and Resistance Factor Design (LRFD), or Allowable Stress Design (ASD)?

☐ SLD (allowable stress, or Service Load Design) ☐ LRFD (Load and Resistance Factor Design)
☐ LFD (Load Factor Design)
☐ Mixed approach (SLD for foundation capacity and LFD or LRFD for load calculation)

SLD: (7) AZ, AR, GA, NM, NC, PR, TX

LRFD: (8) CT, FL, HI, ID, ME, SC, UT, WA

LFD: (6) CA, KS, MA, KS, NC, WA

Mixed: (10)
(a) stated "mixed" only, no explanation: MN, NH, OR, SD, VT
(b) SLD for foundation capacity and LFD or LRFD for load calculation: KY, IL, IA, NJ, TN

30. For structural design purposes, how would you best describe the analysis method used to obtain the distribution of moment and shear with depth?

☐ A "point of fixity" is assumed; shaft is then treated as a structural beam-column.

(11) CT, KS, KY, MA, NJ, NM, NC, SD, TN, TX, UT

☐ Soil/Structure Interaction analysis is conducted using one of the following computer codes:

☐ *p-y* method by COM624 or LPILE^{Plus}

(21) AZ, AR, CA, GA, HI, ID, IL, IA, KS, ME, MA, MN, NJ, NM, OR, PR, SC, TX, UT, VT, WA

☐ FBPIER (5) FL, ME, MN, NC, VT

☐ Other (2) NH, WA (S-Shaft Program see above)

☐ Elasticity solution (1) KS

☐ Numerical methods such as finite element, boundary element, or finite difference specify computer program: *none*

☐ Other method (explain briefly).

NM: Interaction with Geotechnical Section.
TN: Triangular stress distribution limited to side bearing capacity of rock and McCorkle side resistance equations.
TX: Point of fixity is used for simple, "typical" structures. *P-y* method is used for more complex structures.

31. Please indicate whether you have encountered difficulties associated with the design or analysis issues listed below and, if so, summarize the circumstances (shaft dimensions, depth of soil over rock, rock or IGM type):

☐ Unexpectedly high computed shear in the rock socket when using the *p*-y method of analysis.

CA: "When the moments go from a maximum to zero over a relatively short length, then the corresponding shear demands that are reported are large."

☐ Difficulties or questions in applying *p-y* analysis to relatively short socket lengths

CA, IA, NM
NH: "One question is whether the drilled shaft length can be terminated even though the *p-y* analysis indicates some minor shear, moment, or deflection at the base of the shaft."

☐ Questions regarding transfer of moment to the rock socket or development length for reinforcing bars extending into the rock socket.

IA, MA

32. Please identify any other issues pertaining to structural analysis/design that you feel should be addressed in this synthesis.

MA: "Should seismic design of rock-socket length be adequate to develop full plastic hinge moment in reinforced concrete shaft?"
OR: "Not specifically related to rock sockets, but a design with about 60–70 ft of overlying silt was difficult to analyze. Resulting moments at superstructure were opposite direction of what would be expected, did not tend to converge on a solution during seismic modeling runs. I chased it all over the place (using LPILE, WinSTRUDL, ODOT BRIG2D software)."

Part VI: Construction and Field Testing of Rock-Socketed Drilled Shafts

33. Indicate whether the measures described below are included in construction specifications for rock or IGM sockets designed by your agency:

☐ Roughening of the sides of the socket by grooving or rifling (6) AZ, IA, KS, ME, MA, MN

☐ Restrictions on the use of slurry in sockets (14)

CA, FL (no polymer), GA, HI, KS, KY, ME, MA, MN, NM, NC, TX, UT, VT

☐ Specifications for rock excavation by blasting (4) CA, ME, NC, OR

34. Does your agency specify requirements for cleanliness at the bottom of the excavation prior to concrete placement?

☐ Yes (28) AL, AZ, AR, CA, FL, GA, HI, ID, IL, IA, KS, KY, ME, MA, MN, MT, NH, NJ, NM, NC, OR, SC, SD, TN, TX, UT, VT, WA

☐ No (1) CT

If you answered "Yes," please provide a brief summary of the following:

<u>Requirements for cleanliness</u>:

Six states (FL, HI, IL, NH, NC, and SC) gave the following: "minimum 50% of base area to have less than 0.5 in. and maximum depth not to exceed 1.5 in."
<u>AR</u>: No more than 1 in. of loose material.
<u>CA</u>: Specification simply states that the contractor verifies that the bottom is clean.
<u>CT</u>: Not written into specification, but generally following recommendations in FHWA *Drilled Shaft Manual.*
<u>GA</u>: No loose sediment or debris.
<u>ID</u>: Less than 2 in. thick for end bearing shafts; less than 6 in. for side friction shafts.
<u>IA</u>: Minimum 50% of base to have less than 0.5 in. and maximum depth not to exceed 1 in.
<u>KS</u>: Just prior to placing concrete, a minimum of 75% of the base must have less than 0.5 in. of sediment; CSL also used for wet pours.
<u>KY</u>: Maximum 0.5 in. of sediment.
<u>MA</u>: End bearing <1 in., skin friction <3 in.
<u>ME</u>: Minimum of 50% of the base of shaft should have <0.5 in. of sediment at time of concrete placement.
<u>MN</u>: From our drilled shaft special provision: loose material shall be removed from drilled shafts prior to placement of reinforcement. After the shafts have been cleaned, the engineer will inspect the shafts for conformance to plan dimensions and construction tolerances. If permanent casing is damaged and unacceptable for inclusion in the finished shaft, the casing shall be replaced at the contractor's expense. If a portion of a shaft is underwater, the contractor shall demonstrate that the shaft is clean to the satisfaction of the Engineer. This shall include inspection by a diver, at no cost to the department, if considered necessary by the Engineer. Dewatering of the drilled shafts for cleaning, inspection, and placement of reinforcement and concrete will not be required. If the drilled shaft contractor chooses to dewater the shafts for convenience of construction, this work shall be done at the contractor's expense.
<u>NJ</u>: Less than 0.5 in. of sediment.
<u>NM</u>: <1 in. of loose material.
<u>OR</u>: No more than 2 in. of loose material for end-bearing; no more than 6 in. of loose material for friction shafts. Assume end-bearing if not specified.
<u>SD</u>: Make sure the bottom of the shaft is free of loose material.
<u>TN</u>: No loose soil or rock cuttings allowed.
<u>TX</u>: From Specification 416 Drilled Shafts—"remove loose material and accumulated seep water from the bottom of the excavation prior to placing the concrete."
<u>UT</u>: Remove all lose material from the bottom of drilled holes before placing concrete.
<u>WA</u>: The contractor shall use appropriate means such as a cleanout bucket or air lift to clean the bottom of the excavation of all shafts. No more than 2 in. of loose or disturbed material shall be present at the bottom of the shaft just prior to placing concrete for end bearing shafts. No more than 6 in. of loose or disturbed material shall be present for side friction shafts. End bearing shafts shall be assumed unless otherwise noted in the contract. Shafts specified as both side friction and end bearing shall conform to the sloughing criteria specified for end bearing shafts.

Method(s) and tools used to verify cleanliness requirement:

AZ: Visual and hand probe.
AR: Video equipment or person in hole with suitable lighting and ventilation.
CA: Contractor submits a concrete placement plan for approval. Usually, they will use a clean out bucket to clean the bottom of hole. There is inspection of the drilling slurry at bottom of the shaft prior to placing concrete. Caltrans occasionally will verify the bottom of end bearing shafts with a camera.
CT: Probing of the rock-socket bottom.
FL: Probing, sometimes use SID.
GA: Hand cleaned and inspected.
HI: Weighted tape.
ID: Cleaning buckets or air lift.
IA: Weighted tape, camera inspection (rare), sediment deposition "trial run" in open-top clean-out bucket.
KS: Visual on dry pours, sounding (using probes) underwater.
KY: Judgment of inspector.
MA: Weighted rods, visual check by use of cleaning equipment spoils.
ME: Weighted tape and remotely operated cameras.
MN: See above.
NH: Weighted tape or solid rod.
NJ: Sounding by weighted tape.
NM: Weighted tape/sounding.
NC: 1. Visual Inspection. 2. Steel Probe (10 lb weight). 3. SID shaft inspection device.
OR: Weighted tape, rod probe or visual.
SD: Visual inspection if the shaft is dry, otherwise use a clean-out bucket.
TN: Visual inspection.
TX: Visual, clean-out bucket.
UT: Visual.
WA: The excavated shaft shall be inspected and approved by the Engineer prior to proceeding with construction. The bottom of the excavated shaft shall be sounded with an airlift pipe, a tape with a heavy weight attached to the end of the tape, or other means acceptable to the engineer to determine that the shaft bottom meets the requirements in the contract.

35. Does your agency use construction specifications or special provisions that account for construction of sockets in a particular rock type? ☐ Yes ☐ No

Yes: (3) AZ, ID, KY

No: (25)

If "Yes," please provide a brief description.

Rock type and special provision:
AZ: Limestone; drill below tip elevation to check for karst conditions.
ID: Special provisions for IGMs and hard rock.
KY: Soft shale; sometimes require contractors to use polymer slurry in wet holes with soft shales subject to slaking in the presence of water.
CA: Different pay items for Cast-in-Drilled-Hole concrete piling and Cast-in-Drilled-Hole (Rock Socket) concrete piling.

36. Have you observed any methods, equipment, or materials used for socket construction that you believe are a source of construction problems?

☐ Yes ☐ No

Yes: (10) CA, FL, KS, KY, NH, NC, SD, TN, UT, VT

No: (16) AL, AZ, AR, CT, GA, HI, ID, IL, IA, ME, MA, MN, OR, SC, TX, WA

If yes, please explain:

CA: We design for low bidding contractors to get the contract and the construction problems that will result. Rock may be harder than the contractor thought when bidding and planning the job. Thus, the drilling equipment brought out is often unable to drill or very slow to drill the rock. This results in costly contractor claims.

FL: Improper equipment or size; full-length casing reduces skin friction.

KS: In wet pours, inadequate sealed tremie or no pig in the concrete pump supply line. Loss of slump in the concrete during placement. Dirty bottoms were observed with Sonic testing.

KY: Reverse circulation drilling methods used in conjunction with polymer slurry when used as described in Question 35.

NH: Certain clean-out buckets cannot always meet the cleanliness requirements.

NC: Various methods used to force a dry pour.

SD: If not done properly using pump trucks to place concrete can cause soft bottoms in the shafts.

TN: We have significant depths of interlayered rock layers and soil that makes it difficult to use either an auger or a core barrel.

UT: Concerns with use of drilling fluids instead of casing.

VT: In holes cased through the overburden soils into bedrock we have had problems seating the casing into rock. This was true when the contractor used a casing diameter that was essentially the same as the rock-socket diameter. The casing was vibrated into the rock-socket hole, which resulted in more rock drilling than expected, because the casing "followed" the socket. This resulted in longer overall shaft lengths than planned, particularly when the upper portion of the rock was fractured or weathered.

37. Please identify any other construction-related issues for rock or IGM sockets that you believe should be addressed in this synthesis.

FL: What is rock, where does it start, quality.

KS: Pertaining to Question 35—Our special provision accounts for a wet or dry pour (cased or uncased) rather than rock type.

MA: Define "Top of Rock," which generally can be a discrepancy between borings and construction drilling.

NH: Effect of slurry on side friction.

OR: If during construction the top of rock elevation is found to be different than what was assumed in design, what is the effect? How different does it have to be to have a significant effect on design?

38. Indicate whether your agency has used any of the following field load testing methods on rock-socketed drilled shafts.

☐	Conventional static axial load test	(7)	CA, FL, GA, IA, NC, TX, UT
☐	Conventional lateral load test	(5)	CA, FL, MA, NJ, NC
☐	Osterberg Cell for axial load test	(18)	CA, CT, FL, GA, HI, IL, IA, KS, KY, ME, MA, MN, NJ, NM, NC, PR, SC, TX
☐	Osterberg Cell for lateral load test	(1)	SC
☐	Statnamic test for axial load	(6)	CT, FL, IA, NC, PR, SC
☐	Statnamic test for lateral load	(3)	AL, FL, SC

☐ High strain impact (3) FL, KS, MA

If your agency has used the Osterberg Cell (O-cell) for axial load tests on rock-socketed shafts, please answer the following:

39. Were you able to measure both side and tip resistances of the socket independently?

☐ Yes ☐ No

Yes: (17) CA, CT, FL, GA, HI, IL, IA, KS, KY, ME, MA, NJ, NM, NC, PR, SC, TX
No: (1) MN

40. Was the test used to determine

☐ Ultimate side resistance (of socket)
(14) CA, CT, FL, GA, IA, KS, KY, ME, MA, MN, NM, NC, SC, TX

☐ Ultimate base resistance
(8) IL, FL, IA, KS, KY, NC, SC, TX

☐ Proof load only
(4) HI, NJ, PR, SC

41. Additional comments regarding use of O-cell for load testing of rock sockets.

IL: Too expensive.
IA: None.
KY: We have typically failed shafts in side resistance and mobilized enough base movement to extrapolate the ultimate base resistance.
ME: Did not mobilize ultimate base resistance on either project.
MN: Was also used to develop *p-y* curve.
NM: Use was for IGM Santa Fe Formation.
TX: For information on recent O-cell testing in rock contact Dr. Vipu and University of Houston.

If your agency has experience with Statnamic testing of rock-socketed drilled shafts please answer the following:

42. Which of the following performance parameters were determined by the test? (Check all that apply.)

☐ Socket side resistance
(5) FL, AL, IA, NC, SC

☐ Total socket resistance (side and base)
(4) FL, NC, PR, SC

☐ Lateral load-displacement response
(3) FL, NC, SC

☐ Socket base resistance
(5) CT, FL, IA, NC, SC

☐ Axial load displacement response
(5) AL, FL, NC, PR, SC

43. Additional comments regarding Statnamic testing of rock-socketed drilled shafts.

FL: Limit on size that can be tested.
NC: Test is very expensive; we need to find another method with less cost.

44. Do you have results of load tests on rock-socketed drilled shafts and, if so, are you willing to receive follow-up contact regarding the possibility of using your results for the synthesis?

☐ Yes, I have previously unpublished load test results
(9) CA, GA, IA, KS, KY, ME, MA, NC, SC

☐ Yes, I am willing to receive follow-up contact
(11) CA, CT, GA, IA, KS, KY, ME, MA, NM, NC, PR

☐ If previously published, please give a reference:
ME: Loadtest, Inc., performed all O-cell tests on the Bath–Woolwich and Hancock–Sullivan bridges in Maine.
MN: *Transportation Research Record 1633*.

45. Indicate which of the following nondestructive testing methods are used on a regular basis by your agency for rock-socketed shafts.

☐	gamma-gamma	(3) AZ, AR, CA
☐	crosshole sonic logging	(20) AZ, AR, CA, CT, HI, ID, IA, KS, KY, ME, MA, MN, NJ, NM, NC, OR, PR, SC, SD, VT
☐	sonic echo	(1) AR
☐	impulse response	(1) AR
☐	parallel seismic	
☐	other	(3) CA (downhole camera), NC (Osterberg load cell), VT (CSLT—one project)

46. Based on your experience, are there any special considerations or issues related to the use of NDT-NDE, specifically for rock-socketed shafts? If so, explain.

FL: Results are iffy.
IA: None.
KY: No.
MA: The configuration of the test pipes within the socket (if diameter is smaller than shaft) and the possible influence of rock material properties on the data results.
NM: Sonic echo not utilized.
NC: Technology is not 100% accurate.
PR: We bought the equipment (CSL) last month.

47. Do you have case histories of design, construction, or testing of rock-socketed drilled shafts that, in your opinion, could provide useful information to your colleagues and, if so, are you willing to be contacted by the author of the synthesis to discuss your case histories further?

☐ Yes, I have useful case histories (9) CA, CT, IA, KS, KY, ME, NM, NC, WA

☐ Yes, I am willing to receive follow-up contact (8) CA, CT, GA, IA, KS, KY, ME, NM

Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation