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Drilled Shafts: Construction Procedures and Design Methods

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16. Abstract

This manual is intended to provide a technical resource for engineers responsible for the selection and design of drilled shaft foundations for transportation structures. It is used as the reference manual for use with the National Highway Institute (NHI) training course No. 132014 on the subject, as well as the 10th in the series of FHWA Geotechnical Engineering Circulars (GEC). This manual also represents an updated edition of this GEC10 document (FHWA-NHI-10-2016) dated May 2010, co-authored by Dan A. Brown, John P. Turner, and Raymond J. Castelli. This manual embraces both construction and design of drilled shafts, and addresses the following topics: applications of drilled shafts for transportation structure foundations; subsurface investigation specific to information required for construction and design of drilled shafts; construction means and methods; overall design process; geotechnical design of drilled shafts for axial and lateral loading; LRFD structure design; field loading tests; construction specifications; inspection and records; non-destructive integrity tests; acceptance and assessment of drilled shafts, remediation of deficient shafts; and cost estimation.

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- Transportation Research Board

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CHAPTER 1
SELECTION AND USE OF DRILLED SHAFT FOUNDATIONS FOR TRANSPORTATION STRUCTURES

1.1 PURPOSE AND ORGANIZATION OF MANUAL

This technical manual is intended to provide a resource for engineers responsible for the selection and design of drilled shaft foundations for transportation structures and as a reference for use with the three-day National Highway Institute training course on the subject (No. 132014). This document represents an updated edition of this document (publication FHWA-NHI-10-016 by Brown et al.) dated May, 2010. Throughout the document, many photographs and figures have been updated to better illustrate discussion items and reflect recent developments in equipment and techniques for construction and testing. Specific changes to the manual since 2010 also include:

- The document is updated to be consistent with the latest (2017a) AASHTO standards.
- Chapter 2 on site investigation focuses exclusively on guidance specific to information required for construction and design of drilled shafts, with references to other publications for more general information. In particular, the treatment of transitional materials that were previously described as "Intermediate Geo-Materials (IGM)" has been modified, along with a discussion of characterization of rock mass for engineering applications.
- An overview of construction methods is provided in Chapter 3 and is intended to provide a basic explanation of the fundamental principles of drilled shaft construction.
- Chapter 4 (tools and equipment) is updated to reflect the evolution of equipment and techniques since the previous edition.
- Chapter 5 provides a discussion of drilled shaft construction methods with a focus on the use of casings and support fluids to maintain a stable drilled shaft excavation.
- Chapter 6 updates the discussion of reinforcement for the design and construction of drilled shafts with emphasis on the special considerations related to constructability. Updates include bracing systems, considerations for handling, the potential use of new higher strength and weldable steel, and top-of-shaft connection details.
- The updated material on placement and design of concrete for drilled shafts in Chapter 7 presents the significant advances in concrete technology during the last 10 years, including recent and ongoing FHWA sponsored research on concrete for below ground construction.
- Chapter 8 includes a streamlined overview of the drilled shaft design process, providing a roadmap for design with a step by step description of the tasks to be completed.
- Design for lateral loading in Chapter 9 is updated and shortened. With the publication of Geotechnical Engineering Circular 9 to cover the design of all types of deep foundations for lateral loading, this chapter is now focused on those aspects particular to drilled shafts. This chapter includes specific information on the design of drilled shafts for seismic loading, a topic which has evolved over the last decade.
- In Chapter 10, design equations for axial loading are updated for base resistance in cohesionless soils and side resistance in cohesive soils. Revised methods are presented for axial design under earthquake incorporating recent research on pore pressure generation, liquefaction, and post-liquefaction downdrag.

- Design of groups of drilled shafts (Chapter 11) is updated to reflect recent changes to AASHTO design guidelines for evaluation of drilled shaft groups for strength limit.
- Structural design (Chapter 12) now includes a method for structural design of rock sockets using strut-and-tie model (STM) in accordance with the 2017 AASHTO LRFD Bridge Design Specifications. Additional minor revisions have been incorporated to reflect changes in Section 5 (Reinforced Concrete) of the AASHTO LRFD Bridge Design Specifications.
- Chapter 13 on field load testing is updated to provide more guidance on numbers and locations of drilled shaft load tests for transportation projects, and incorporates some illustrative examples. The material in this chapter is also updated to include new technology in instrumentation and measurement.
- The discussion of guide specifications in Chapter 14 is updated to include references to new ASTM standards and technology, and include new discussion of issues related to measurement and payment provisions. The actual guide specifications are relocated to Appendix D.
- The chapter on inspection and records (Chapter 15) has updates related to pilot hole requirements, concrete volume plots, and an updated checklist.
- Chapter 16 on integrity testing includes a discussion of thermal integrity profiling (TIP), which has become a widely accepted method of nondestructive testing for completed drilled shafts.
- Chapter 17 outlines a process for assessment and acceptance of completed drilled shafts based on inspection records and integrity testing. A procedure is outlined for the steps required when further evaluation is needed in response to construction observations or testing anomalies. Chapter 17 also discusses available methods for drilled shaft remediation when it is determined that defects require repair, illustrated with case histories.
- Chapter 18 has been updated with more recent information affecting cost estimation.

This manual is intended to provide guidance with respect to recommended practice for general design and construction of drilled shafts for transportation facilities in the U.S. It does not include comprehensive coverage of every design and construction method which might be employed, nor does it reflect current practice for other types of infrastructure, such as building foundations. Local practice adapted to unusual circumstances or to specific local geologic conditions may evolve differently from some specific recommendations outlined in this manual. Although the recommendations given in this publication represent generally recommended practice as of the time of this writing, it is not intended to preclude deviations from these recommended practices that are based on demonstrated performance and sound engineering.

The manual is organized in several major sections consistent with the presentation of the short course materials. The major sections include:

- **Overview and Applications.** This chapter (1) provides an overview of drilled shaft foundations, along with a discussion of general applications of the technology for transportation structures. This information is intended to provide a general basis for understanding the subsequent chapters and to aid designers in identifying those applications for which drilled shaft foundations might be appropriate.
- **Needed Geotechnical Information.** Chapter 2 describes the aspects of geotechnical site characterization and determination of material properties specifically required for construction and design of drilled shafts.
- **Construction.** Chapters 3 through 7 explain construction methods and materials. Chapter 3 describes the overall construction process, with more detailed discussion of specific aspects in Chapters 4 through 7. It is very important that design professionals understand construction of drilled shafts to produce constructible and cost-effective drilled shaft designs.

- **Design.** Chapters 8 through 12 present guidelines for design of drilled shafts for axial and lateral loadings using the principles of LRFD based design.
- **Quality Assurance.** Chapters 13 through 18 cover issues relating to specification, inspection, testing, and quality assurance, as well as cost estimation.

The manual also includes a comprehensive example of a bridge foundation designed with drilled shafts. The example, illustrated below in Figure 1-1, is an intermediate pier of a bridge across a river, with the bent supported by three columns. The new bridge replaces an adjacent existing structure founded on driven piles. The drilled shaft design is to consist of an individual shaft supporting each of the three columns. Details of the project requirements, subsurface information, and foundation design are presented in total in Appendix A.

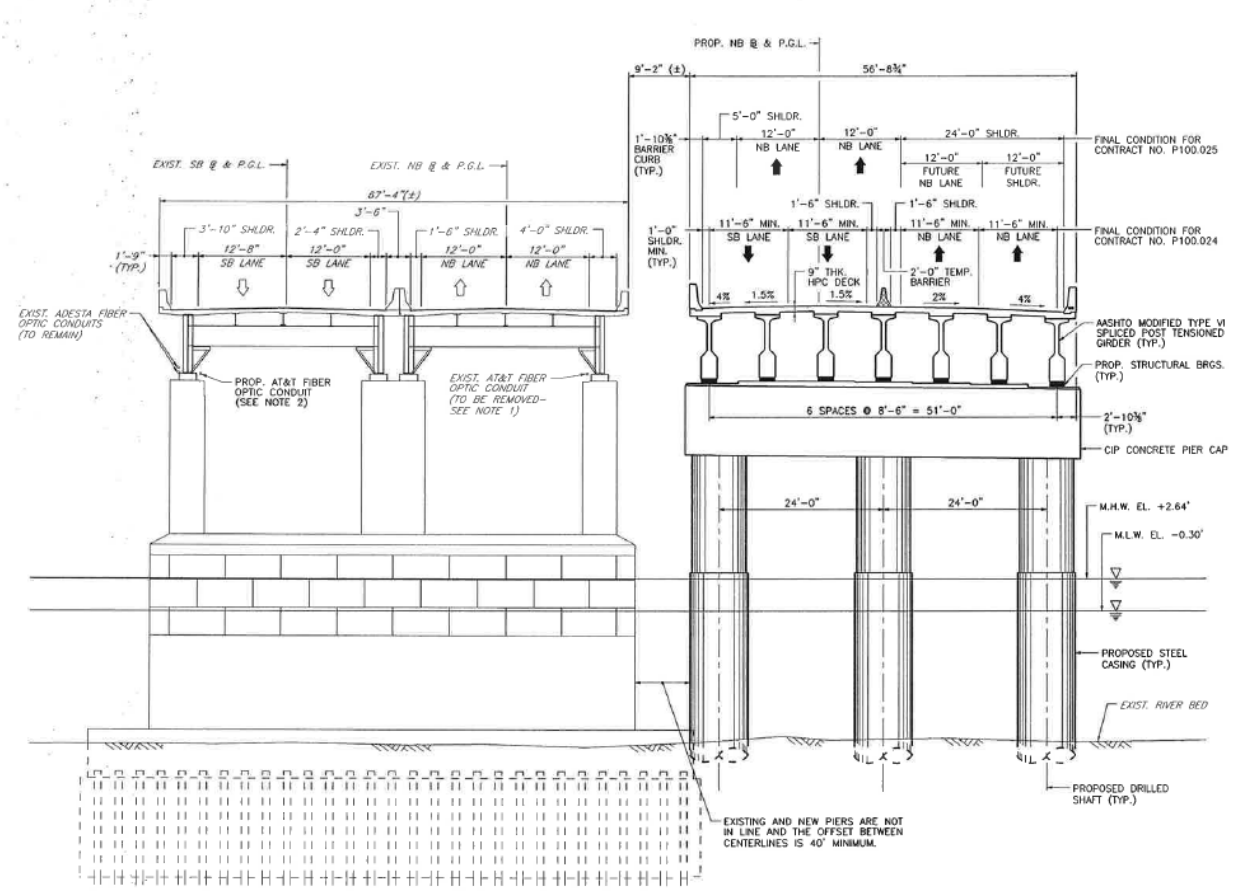


Figure 1-1 Design Example (see Appendix A).

1.2 DEEP FOUNDATION ALTERNATIVES

Drilled shaft foundations are broadly described as cast-in-place deep foundation elements constructed in a drilled hole that is stabilized to allow controlled placement of reinforcing and concrete. Several other types of deep foundations are employed in transportation works, as described below with distinctions from drilled shafts.

- Driven piles are prefabricated structural elements which are installed into the ground with a pile driving hammer. Driven piles have been used to support structures for thousands of years and in present times steel H, pipe, and prestressed concrete piles are commonly used for transportation structures. Guidelines for design and construction of driven pile foundations are provided in

FHWA NHI-16-009 (Hannigan et al., 2016). Driven piles are typically 12 to 60 inches in diameter or width, and thus smaller in size than drilled shafts. Driven piles displace the soil into which they are driven and typically cannot penetrate hard materials or rock. In soft or caving soils there is no concern for stability of a hole.

- Micropiles are drilled piles which are typically less than 12 inches diameter and constructed using a high strength steel rod or pipe which is grouted into the bearing formation. Guidelines for design and construction of micropiles are provided in FHWA-NHI-05-039 (Sabatini et al., 2005). These piles can be drilled into even hard rock and achieve very high axial resistance for a very small structural member. Micropiles are favored in conditions where the small size is an advantage and where lightweight, mobile drilling equipment must be employed.
- Continuous flight auger piles and drilled displacement piles are drilled pile foundations which are typically 12 to 30 inches in diameter. These piles are distinguished from drilled shafts in that the pile is formed by screwing the continuous auger or displacement tool into the ground and then grouting or concreting through the hollow center of the auger; thus there is not an open hole at any time during the construction process. Guidelines for the design and construction of these types of piles are provided in FHWA GEC-8 (Brown et al., 2007).

All of the types of piles described above are most often used in groups connected at the pile top with a reinforced concrete pilecap. Drilled shafts are distinguished from other types of piles in that drilled shafts are often substantially larger in size, frequently used as a single shaft support for a single column without a cap, and often installed into hard bearing strata to achieve very high axial and lateral load resistance in a single shaft. A description of drilled shafts and applications which may favor the use of a drilled shaft foundations follows.

1.3 DRILLED SHAFT FOUNDATIONS – DESCRIPTION AND HISTORY

Drilled shaft foundations are formed by excavating a hole, typically 3 to 12 feet in diameter, inspecting the soil or rock into which the foundation is formed, and constructing a cast-in-place reinforced concrete foundation within the hole. The foundation, as constructed, supports axial forces through a combination of side shear and end bearing resistance. The large diameter reinforced concrete member is also capable of providing substantial resistance to lateral and overturning forces as illustrated on Figure 1-2. Drilled shafts for transportation structures are now commonly used to depths of up to 200 ft in the U.S., but can extend to depths of as much as 300 ft or more.

Drilled shafts are also referred to by other names, including drilled piers, caissons, cast-in-drilled-hole piles (CIDH - Caltrans terminology), and bored piles (Europe). The common reference to these foundations as “caissons” reflects the history of development of drilled shaft foundations.

The term “caisson” is more accurately used to reference very large footings which are sunk into position by excavation through or beneath the caisson structure, and the use of drilled shafts evolved in many respects from caisson construction. Caisson construction has been used for hundreds of years, and was pioneered in the U.S. bridge construction in 1869 by James Eads in St. Louis and subsequently in the 1870’s by Roebling on the Brooklyn Bridge (McCullough, 1972). A diagram of caisson construction is shown on Figure 1-3 from one of the world’s most famous bridges, the Firth of Forth crossing in Scotland. These caissons were constructed as “pneumatic caissons” in which air pressure was maintained below the caisson as it sunk to prevent water inflow into the chamber below where workers excavated beneath the caisson cutting edge to sink the caisson to the required bearing stratum. Pneumatic caissons are rare today because of safety issues, but open well caissons are still occasionally used for bridges in deep water environments.

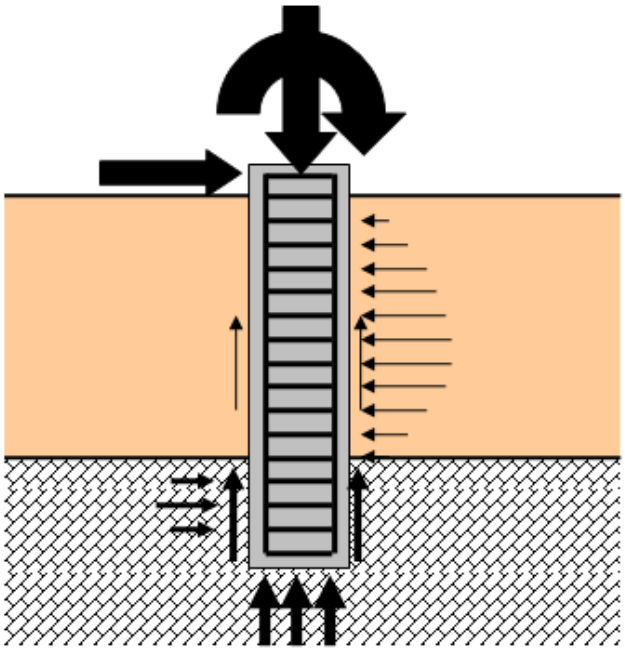


Figure 1-2 Schematic of Axial and Lateral Resistance of a Drilled Shaft

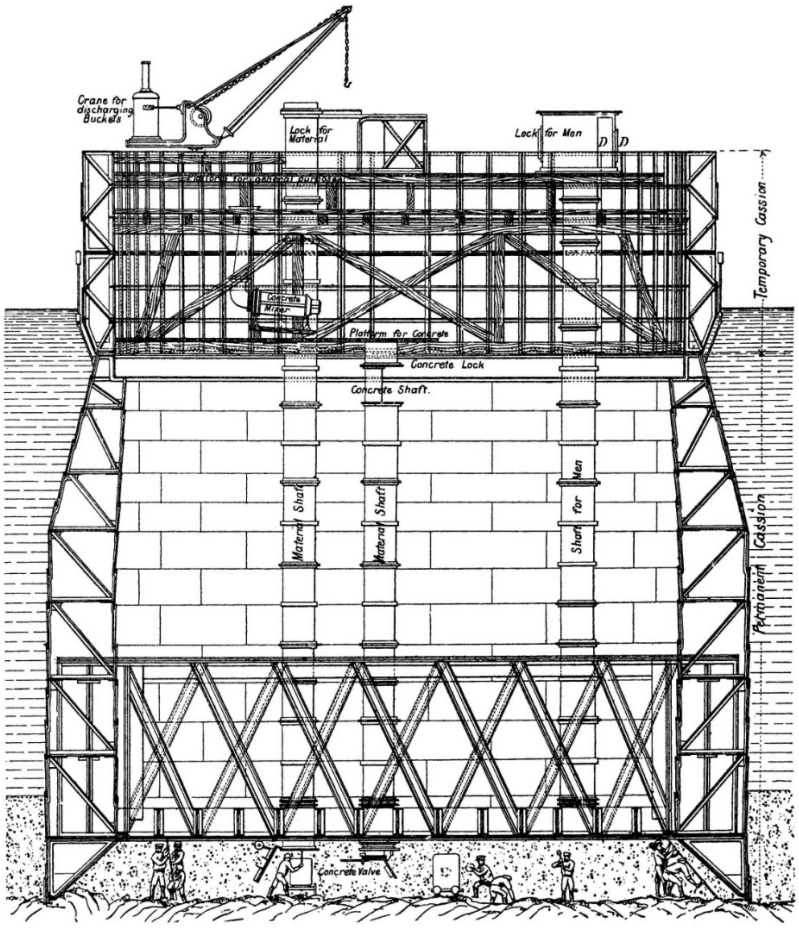


Figure 1-3 Pneumatic Caisson for Firth of Forth Bridge (Mackay, 1990)

Open well caissons typically consist of a box open at both top and bottom, with dredge wells for excavating the soil through the caisson to sink it into place. Several large bridges have recently been constructed on large rectangular "open-well" caissons including the new Tacoma Narrows bridge and the Mississippi River crossing at Greenville, MS (Figure 1-4).

Small, circular caissons or shafts were used to support building structures and some transportation structures in the early 1900's in several large cities including Kansas City, Chicago, Boston, and New York. These early forms of drilled shafts were usually excavated by hand. The first known building supported on caissons of this type is the City Hall in Kansas City, which was constructed in 1890 (Hoffmann, 1966). Because of concern that timber piles might rot, the city building superintendent, S.E. Chamberlain designed the foundations to consist of 92 caissons, 4½-ft diameter, placed to bear on limestone at a depth of around 50 ft. The excavation was supported by cylindrical sections of 3/16" boiler plate "to prevent the collapse of earth surrounding the excavation" (Chamberlain, 1891), and backfilled not with concrete but with vitrified brick laid in hydraulic cement. A drawing from the Kansas City Star newspaper is reproduced in Figure 1-5. Chamberlain's description of this approach at the Annual Convention of the American Institute of Architects in Chicago in the fall of 1890 may have contributed to the adoption of this technique for several structures in that city soon afterward.

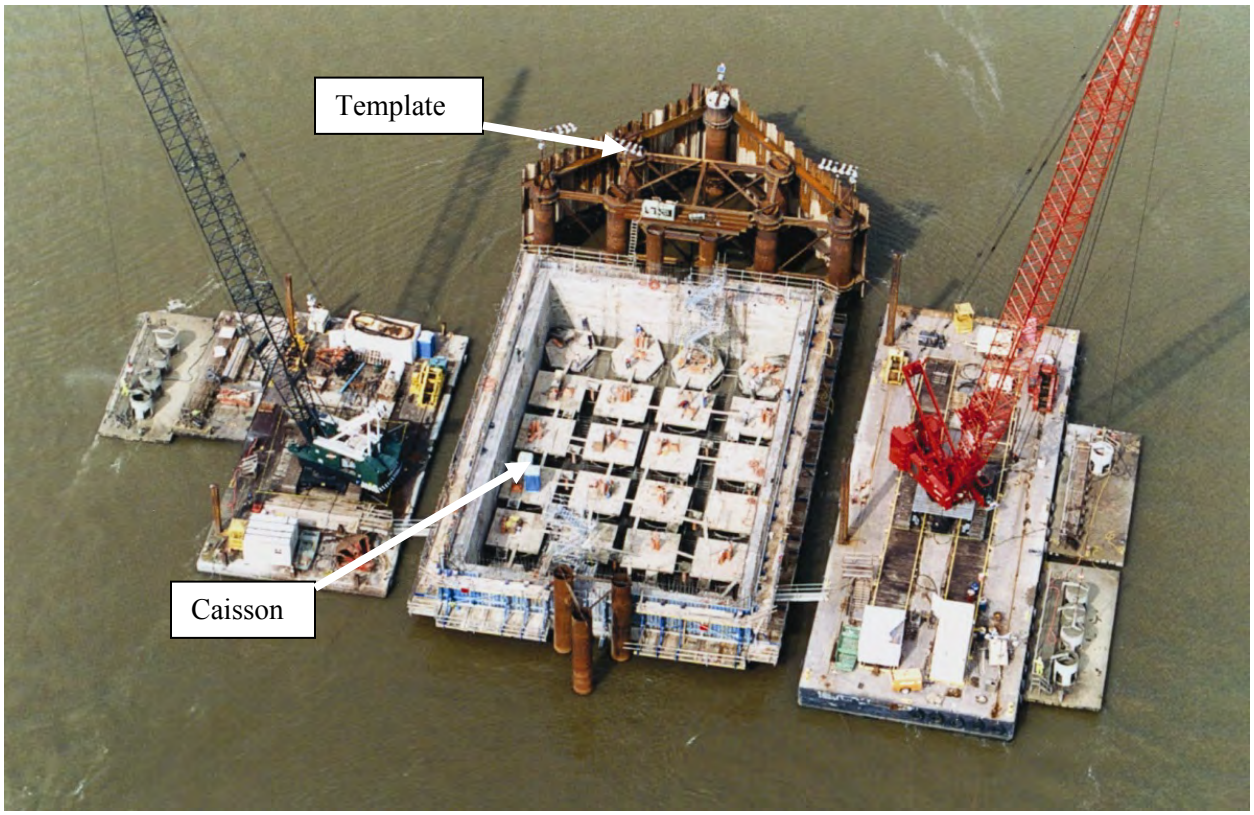


Figure 1-4 Caisson Construction for Greenville Bridge (Jacobson, 2007)

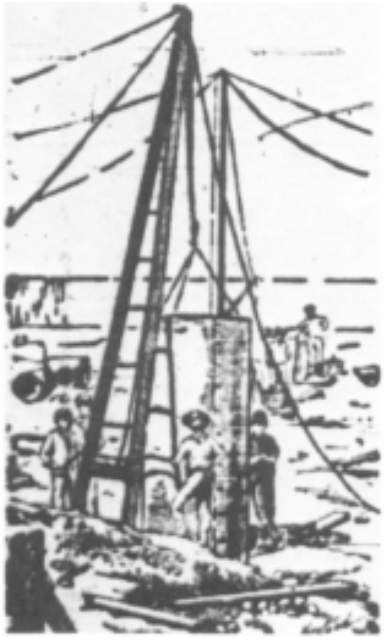


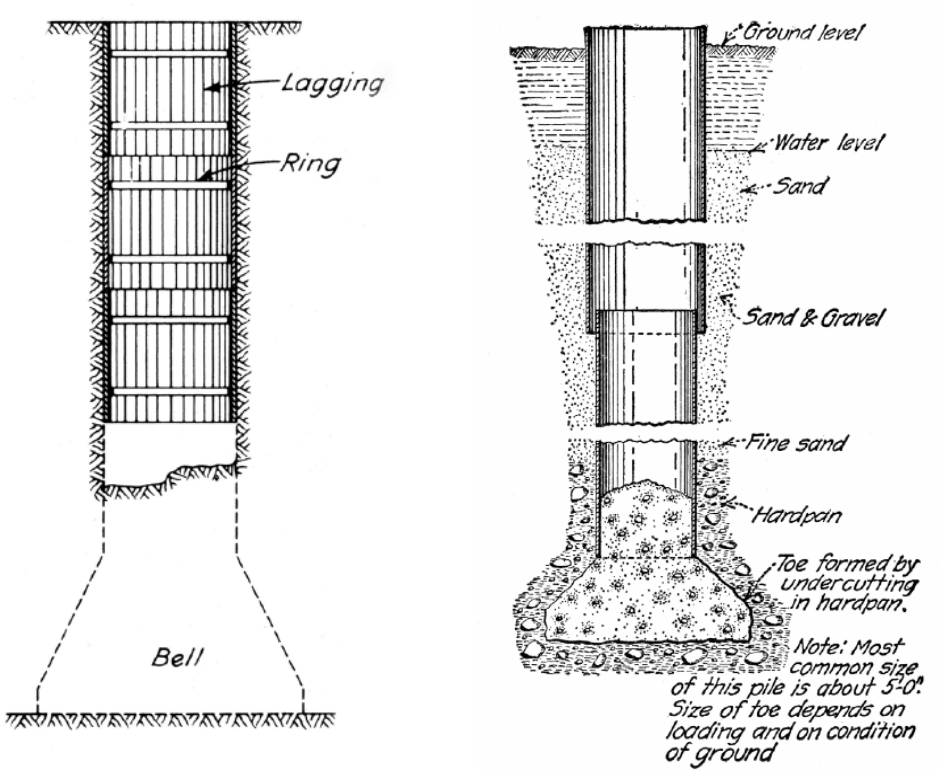
Fig. 2. Caisson being lowered into place for the City Hall, drawing from the *Star*, 19 September 1892 (courtesy State Historical Society of Missouri).

Figure 1-5 "Caisson" Foundation Construction in Kansas City (Hoffmann, 1966)

Several notable buildings in Chicago which had been founded on spread footings had suffered damaging settlement. The use of timber piles caused such heaving of the surrounding area that the owners of the Chicago Herald got a court injunction to stop construction of the pile foundations at the Chicago Stock Exchange building because of structural damage to their building (Rogers, 2006). The diagram in Figure 1-6(a) illustrates a foundation of the type designed by William Sooy Smith for one wall of the Chicago Stock Exchange building in 1893. The shafts were constructed as circular excavations with tongue and grooved timber lagging which was driven ahead of the excavation and braced with iron hoops. This method of excavation with timber lagging in a circular form became known as the "Chicago Method." These types of foundations are not actually caissons in the true sense of the word, but the term stuck and is still used today even for modern drilled shaft construction.

The diagram of Figure 1-6(b) illustrates a "Gow caisson" of the type pioneered by Col. Charles Gow of Boston who founded the Gow Construction Co. in 1899. The telescoping casing forms could be recovered during concrete placement. In the 1920's, the Gow Company built and used a bucket-type auger machine which was electrically powered and mounted on the turntable frame of the crawler tractor of a crane (Greer and Gardner, 1986), thus promoting the development of machine-drilled shafts.

There has been a significant evolution of the drilled shaft industry over the past 50 years to the type of construction and design which is prevalent today (2018); machine drilled shafts became more widespread during the 1930's and became increasingly used during the building boom after World War II. The A.H. Beck Company began using drilled shafts in 1932 (photo in Figure 1-7) and, along with McKinney Drilling (founded 1937), were some of the pioneers of the drilled shaft industry in Texas. Augered uncased holes smaller than 30-inch diameter were common, and sometimes tools were employed to rapidly cut an underream or bell. In California, "bucket-auger" machines were more common, using a bottom dumping digging bucket to dig and lift soils rather than an auger.



(a) (b)

Figure 1-6 Early "Caisson" Foundations (Rogers, 2006); (a) "Chicago Method," and (b) "Gow Caisson



Figure 1-7 An Early Drilled Shaft Rig and Crew (courtesy A.H. Beck Foundation Co., Inc.)

Modern drilled shaft construction techniques are described in Chapters 3 through 7 of this manual and include equipment ranging from simple truck mounted equipment used to auger holes not much different from that used in the 1940's, to modern machines capable of drilling large, deep shafts into very hard materials (Figure 1-8). Underwater concrete placement is now commonly employed so that a dry excavation, and worker entry into the excavation, is not required. Techniques for testing to verify geotechnical strength and structural integrity are common so that drilled shafts can be used with a high degree of confidence in the reliability of the foundation.



Figure 1-8 Typical Modern Drilled Shaft Rigs

The photo in Figure 1-9 shows construction of the main pylon foundation for a new cable-stayed Missouri River bridge in Kansas City, 108 years after the pioneering the first use of drilled shafts for the City Hall. The equipment and construction methods have advanced far beyond the original concepts proposed in 1890, but the basic idea is the same: to support the structure on bedrock below weak soils using small, economically constructed "caisson" foundations. The history of drilled shafts is thus seen to have come full circle: the large caisson construction techniques used for bridges were adapted to construct small diameter "caissons" to support buildings which lead back to the use of large drilled shafts for bridges and other transportation structures.

The development of improved equipment, materials, and methods for design and testing have allowed the cost effective use of drilled shafts in a greater variety of applications and with greater reliability than was ever before possible. The following sections provide an overview of some applications of drilled shafts for transportation structures along with factors affecting the selection of drilled shafts as a deep foundation alternative.

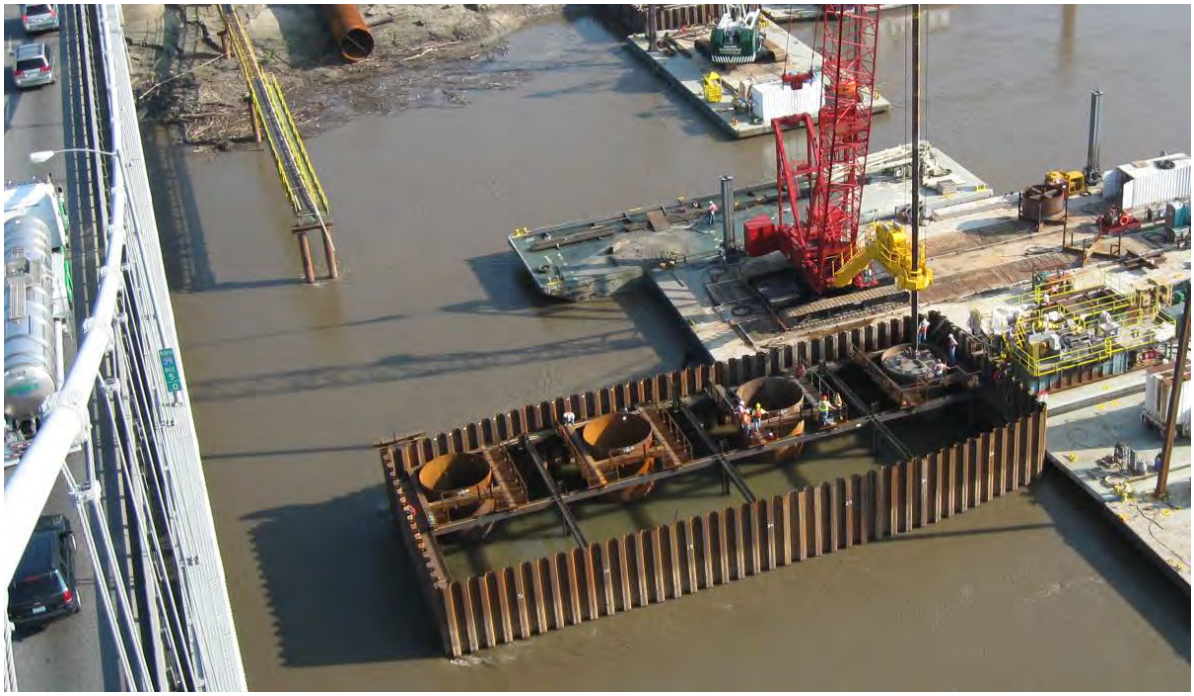


Figure 1-9 Construction of 12-ft Diameter Drilled Shafts for the Main Pylon Foundation, Christopher S. Bond Bridge, Kansas City (photo by Gary Naugle, MoDOT)

1.4 SELECTION OF DRILLED SHAFTS

Drilled shafts can be installed in a variety of soil and rock profiles, and are most efficiently utilized where a strong bearing layer is present. When placed to bear within or on rock, extremely large axial resistance can be achieved in a foundation with a small footprint. The use of a single shaft support avoids the need for a pile cap with the attendant excavation and excavation support, a feature which can be important where new foundations are constructed near existing structures. Foundations over water can often be constructed through permanent casing, avoiding the need for a cofferdam. Drilled shafts can also be installed into hard, scour-resistant soil and rock formations to obtain support below scourable soil in conditions where installation of driven piles might be impractical or impossible. Drilled shafts have enjoyed increased use for highway bridges in seismically active areas because of the flexural strength of a large diameter column of reinforced concrete. Drilled shafts may be used as foundations for other applications such as retaining walls, sound walls, signs, or high mast lighting where a simple support for overturning loads is the primary function of the foundation.

The following sections outline some applications for the use of drilled shaft foundations in transportation structures, followed by a discussion of the advantages and limitations of drilled shafts relative to alternative foundation types.

1.4.1 Applications

Drilled shaft foundations are a logical foundation choice for a variety of transportation structures if the loading conditions and ground conditions are suitable. The following sections outline some of the circumstances where drilled shafts are often the foundation of choice for structural foundations.

1.4.1.1 Bridge Foundations

For foundations supporting bridge structures, conditions favorable to the use of drilled shafts include the following:

- Cohesive soils, especially with deep groundwater. For these soil conditions, drilled shafts are easily constructed and can be very cost effective (Figure 1-10).
- Stratigraphy where a firm bearing stratum is present within 100 ft of the surface. Drilled shafts can provide large axial and lateral resistance when founded on or socketed into rock or other strong bearing strata.



Figure 1-10 Construction of Drilled Shaft in Dry, Cohesive Soils

- Construction of new foundations where a small footprint is desirable. For a widening project or an interchange with "flyover" ramps or other congested spaces, a single drilled shaft under a single column can avoid the large footprint that would be necessary with a group of piles. A single shaft can also avoid the cost of shoring and possibly dewatering that might be required for temporary excavations. Construction of drilled shafts can often be performed with minimal impact on nearby structures. Figure 1-11 illustrates some examples of these types of applications. The upper photo shows a drilled shaft constructed in the small space between and existing highway bridge and a rail bridge. The lower photo illustrates construction of a drilled shaft to support a column for the widening of an existing elevated bridge structure.
- Construction of foundations over water where drilled shafts may be used to avoid construction of a cofferdam. The photo in Figure 1-12 illustrates a two column pier under construction in a river with a single shaft supporting each column.
- Foundations with very high axial or lateral loads. The photo in Figure 1-13 shows construction of a 5-shaft group with a waterline footing for a bridge with large foundation loads in relatively deep water.

- Foundations with deep scour conditions where driven piles may be difficult to install. The photos in Figure 1-14 are from a bridge in Arizona. The original piles had been driven to refusal but subsequently one of the foundations had been lost due to scour.
- Construction of new foundations with restricted access or low overhead conditions. Often, high capacity drilled shaft foundations can be constructed with specialty equipment to avoid overhead utilities or existing structures. Construction of new foundations for a replacement structure in advance of demolition of existing structures can be used to reduce the impact of construction on the traveling public. The photo in Figure 1-15 shows a drilled shaft rig designed to work with restricted headroom.



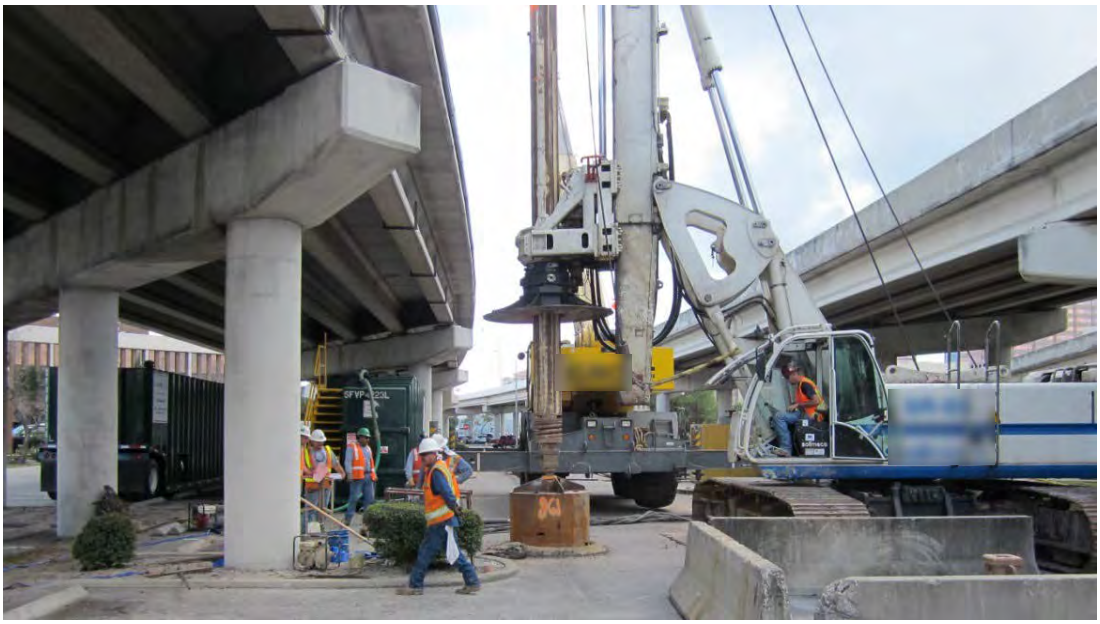


Figure 1-11 Drilled Shafts for Bridge Foundations where Small Footprint is Desirable



Figure 1-12 Drilled Shafts for Individual Column Support over Water

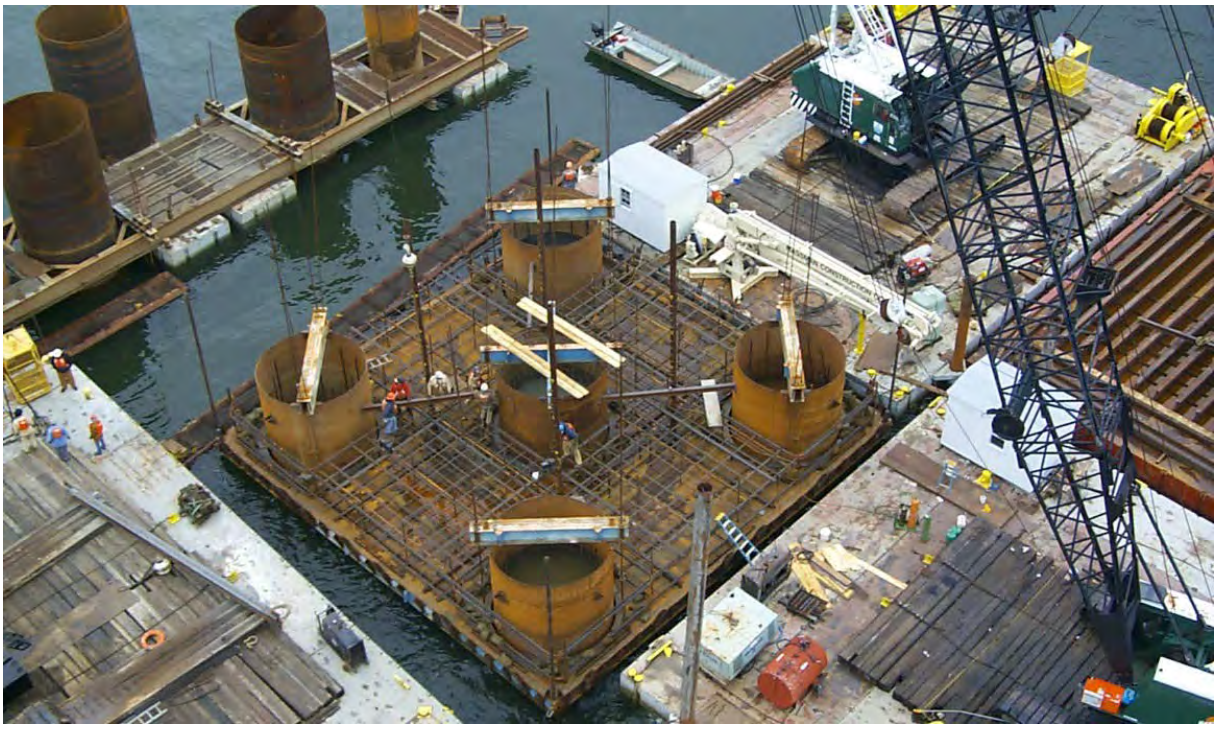


Figure 1-13 Group of Drilled Shafts for Large Loads



Figure 1-14 Drilled Shafts Installed for Deep Scour Problem



Figure 1-15 Drilled Shafts with Low Headroom

1.4.1.2 Other Applications

Drilled shaft foundations can be particularly well adapted to use for other types of transportation structures. Due to the high strength in flexure provided by large diameter reinforced concrete columns, drilled shafts are well suited to structures where loads are dominated by overturning, such as sound walls, signs, and lighting structures. In most cases, drilled shafts for these applications do not require great length and the drilling equipment used to install them may be relatively light and mobile. Some illustrations of these types of applications are provided in Figure 1-16. The design of drilled shafts for lateral and overturning forces is described in Chapter 9 of this manual.

Retaining structures may be founded on drilled shafts, or even may incorporate drilled shafts as the wall itself. Conventional reinforced concrete walls may include drilled shafts supporting the wall footing, but often the shafts can be used in a single row without a footing cap and with the wall cast atop the row of shafts as illustrated on the left of Figure 1-17. Drilled shafts can also be used as soldier beams with precast panels placed between to form a wall as shown on the right.



Figure 1-16 Drilled Shafts for Soundwall (left) and Sign Tower (right)



Figure 1-17 Drilled Shafts Used to Support Earth Retaining Structures

Drilled shafts can even be used to form a top-down wall prior to excavation as a secant or tangent pile system. Secant piles are formed by installing every second shaft without reinforcement, then subsequently drilling the remaining shafts with an overlap that cuts into the existing ones and is reinforced to resist lateral earth and groundwater pressures as well as surcharge loads. An example is shown in Figure 1-18; the reinforcement of every second pile is observed extending from the top, where these bars will tie into a cap beam. Tangent pile walls, formed with drilled shafts constructed adjacent to each other or with a small space between without cutting an overlap, may be suitable if a watertight wall is not required. An aesthetic facing (precast concrete panels, or cast-in-place concrete), or shotcrete covering may be used after excavation to expose the wall.

Drilled shafts have been used in landslide stabilization schemes. A drilled shaft wall or even rows of shafts with space between rows can be constructed across a slip surface to provide a restraining force to a sliding soil mass. Although this approach can be an expensive solution to slope stability problems, there may be applications where right-of-way or other constraints preclude grading or changes in slope geometry.



Figure 1-18 Drilled Shaft Secant Wall

1.4.2 Advantages and Limitations

Many of the advantages of drilled shafts are apparent from a review of the applications cited above. In addition, drilled shafts offer the opportunity to directly inspect the bearing material so that the nature of the bearing stratum can be confirmed. However, there is no direct measurement that can be related to axial resistance as in the case of pile driving resistance. The most significant of the limitations are related to the sensitivity of the construction methods to ground conditions, the influence of ground conditions on drilled shaft performance, and the importance of maintaining the necessary standard of care during installation of the drilled shafts. A summary of advantages and limitations of drilled shafts compared to other types of deep foundations is provided below.

ADVANTAGES AND LIMITATIONS OF DRILLED SHAFTS

Advantages	Limitations
• Easy construction in cohesive materials, and most rock	• Construction is sensitive to groundwater or difficult drilling conditions
• Suitable to a wide range of ground conditions	• Performance of the drilled shaft may be influenced by the method of construction
• Visual inspection of bearing stratum	• No direct measurement of axial resistance during installation as with pile driving
• Possible to have extremely high axial resistance	• Load testing of high axial resistance may be challenging and expensive
• Excellent strength in flexure	• Structural integrity of cast-in-place reinforced concrete member requires careful construction, QA/QC
• Small footprint for single shaft foundation without the need for a pile cap	• Single shaft foundation lacks redundancy and must therefore have a high degree of reliability
• Low noise and vibration, and therefore well suited to use in urban areas and near existing structures	• Requires an experienced, capable contractor, usually performed as specialty work by a subcontractor
• Can penetrate below scour zone into stable, scour-resistant formation	• May not be efficient in deep soft soils without suitable bearing formation
• Can be easily adjusted to accommodate variable conditions encountered in production	• Requires thorough site investigation with evaluation of conditions affecting construction; potential for differing site conditions to impact costs, schedule

1.5 KEYS TO SUCCESSFUL USE OF DRILLED SHAFTS

Because drilled shafts are sensitive to the ground conditions and construction techniques used, it is critically important that designers be familiar with these factors so that drilled shafts are selected for use in the appropriate circumstances, the design is constructible, and the specifications and quality assurance measures are suitable to ensure that a reliable foundation is constructed. The keys to successful construction and design of drilled shafts are outlined below.

- **Subsurface Investigation.** A thorough subsurface investigation is required not only for the design of drilled shaft foundations but also for the selection of appropriate construction tools and methods. Issues related to construction of drilled shafts should be addressed at the time of the site investigation and in the subsequent geotechnical report. Items such as groundwater level, relative soil permeability, rock hardness, natural and manmade obstructions, and geologic features which may affect drilling are important for planning and executing the work. Because drilled shafts are often designed to bear in hard soils or rock, characterization of these materials for design purposes can be challenging. Guidelines for site investigation and determination of geomaterial properties are described in Chapter 2.
- **Knowledge of Construction Techniques.** To design drilled shafts which are constructible, cost effective, and reliable, it is essential that engineers and project managers have a thorough understanding of construction methods for drilled shafts. Drilled shaft construction methods and materials are described in Chapters 3 through 7.

- **Design for Constructability and Reliability.** The performance of drilled shafts can be strongly influenced by construction methods and, accordingly, a recurring theme throughout the design process is that constructability be considered at each step. A robust design is one which is simple to execute and construct, and can accommodate variations in subsurface conditions while minimizing risk of delays or costly changes. Design aspects are described in Chapters 8 through 12. The recent development of advanced techniques for load testing drilled shafts has allowed designers to incorporate site specific testing to measure performance, confirm design methodology, reduce risk of poor foundation performance, and avoid designs which are excessively conservative, expensive, and more difficult to construct. Load testing is described in Chapter 13. Constructible designs are more cost-effective; factors affecting cost are summarized in Chapter 18.
- **Appropriate Specifications.** Specifications set forth the contractual rules for execution of the work and must include provisions which are constructible, provide the required means of quality assurance, and achieve the required performance of the completed project. An understanding of construction techniques and the potential influence of construction must be incorporated into specifications which are appropriate for each specific project; guidance for development of drilled shaft specifications is provided in Chapter 14. Guide specifications with commentary are provided in Appendix D.
- **Quality Assurance.** Drilled shaft foundations are cast-in-place reinforced concrete structures which are sometimes constructed under difficult circumstances. In order to ensure that reliable foundations are constructed, a rigorous program of inspection and testing is essential. A discussion of inspection and tests for completed shafts is provided in Chapters 15 and 16. Assessment and acceptance of drilled shafts is discussed in Chapter 17 along with a description of possible remediation techniques for repair of defects.

1.6 SUMMARY

This chapter provided an introduction to drilled shaft foundations, along with a brief history of the development of drilled shafts in the U.S. Some potential applications of drilled shafts for transportation structures are presented, with advantages and limitations compared to other types of deep foundations. Finally, a summary of the keys to successful use of drilled shaft foundations emphasizes the theme of design for constructability. Details on the selection, design, construction, and inspection of drilled shafts are presented in the following chapters. The design of drilled shafts in this manual is presented in the format of load and resistance factor (LRFD) design, consistent with the current (2017) AASHTO standards.

CHAPTER 2
SITE CHARACTERIZATION

2.1 INTRODUCTION

Site characterization is the process of defining subsurface soil and rock units and their physical and engineering properties. For drilled shafts, site characterization information is used for two general purposes: (1) analyzing shaft resistance and load-deformation response for design, and (2) evaluating construction feasibility, costs, and planning. A well planned site characterization makes it possible to design reliable, economical, and constructible drilled shaft foundations that will meet performance expectations. Inadequate site characterization can lead to uneconomical designs, costly construction disputes and claims, and foundations that fail to meet performance expectations. This chapter describes information requirements specific to the design and construction of drilled shafts, considerations for developing appropriates scopes for site characterization investigations, and special site characterization requirements for drilled shafts to promote effective selection, design, and construction. Means and methods for characterizing sites and for establishing important geotechnical design parameters are not addressed in the chapter. Comprehensive guidance regarding means and methods for conducting site investigations is provided in the AASHTO Subsurface Investigations Manual (AASHTO, 1988). FHWA's Geotechnical Engineering Circular No. 5 (GEC 5) – Geotechnical Site Characterization (Loehr, et al., 2017) provides comprehensive guidance for planning and executing site characterization investigations, for establishing appropriate values for design parameters, and for documenting and communicating results from site characterization investigations. These references should be consulted along with the guidance provided in this chapter when planning and executing site characterization for projects involving drilled shafts.

2.2 INFORMATION REQUIRED FOR DESIGN OF DRILLED SHAFTS

Effective site characterization programs should provide all information needed to develop practical and efficient designs for drilled shafts for costs that are commensurate with the risks involved. Table 2-1 summarizes site characterization information that is commonly needed for design of drilled shafts. The required information is divided into four general categories: (1) subsurface stratigraphy, (2) groundwater conditions, (3) classification of soil and rock, and (4) design parameters. Additional information required specifically for constructability is described in Section 2.3.

Stratigraphy is important for design because it often drives considerations of feasibility and because it directly influences the design resistance and performance of drilled shafts, as well as the means and methods of drilled shaft installation. Site characterization programs should therefore identify the types of soil and/or rock present at a site, separate the subsurface into discrete strata, and characterize how these strata vary across the site. Interpretations of stratigraphy are commonly derived from observations and measurements from borings and in situ test soundings. However, geophysical surveys can often be effectively used to improve interpretations of stratigraphy, especially when combined with information from borings and in situ test soundings. Seismic refraction and electrical resistivity methods can be particularly effective for helping to interpret depth to rock, which is often an important consideration for drilled shafts used to support bridges and other transportation structures.

Accurate knowledge of groundwater conditions is needed for drilled shaft design in order to properly assess the state of effective stress. Effective stress methods are used for evaluating side and tip resistance in cohesionless soils for design under axial loading (Chapter 10), for computing *p*-*y* curves for design

under lateral loading (Chapter 9), and for assessing liquefaction of soil deposits under earthquake loading (Chapter 9). Information regarding groundwater conditions is often derived from observing groundwater levels in exploratory borings, especially when groundwater conditions are relatively straightforward (e.g., hydrostatic groundwater conditions) and when water levels in borings respond quickly to conditions in the surrounding ground. However, for more complex conditions or when water levels in borings do not respond quickly, groundwater monitoring wells and piezometers, as well as in situ measurements of pore water pressure (e.g., from piezocone soundings) are necessary to accurately characterize conditions. Groundwater conditions are also commonly time varying, so it is important that measurements made for interpreting groundwater conditions provide some assessment of the expected magnitude of variations over time.

TABLE 2-1 SUMMARY OF INFORMATION NEEDED FOR DRILLED SHAFT DESIGN

Information	Parameter	Subsurface Material		
		Coarse-grained Soils	**Fine-grained Soils**	**Rock**
Stratigraphy	-	X	X	X
Groundwater	-	X	X	X
Classification	Gradation	X	X	-
	Atterberg Limits	-	X	-
	Moisture Content	X	X	-
	Unit Weight	X	X	X
	RQD and Core Recovery	-	-	X
	Slake Durability Index	-	-	X
Design Parameters	Effective Stress Friction Angle	X	X	-
	Undrained Shear Strength	-	X	-
	Preconsolidation Stress	X	X	-
	Soil Modulus	?	?	-
	Uniaxial Compressive Strength	-	-	X
	Intact Rock Modulus	-	-	X
	Rock Mass Modulus	-	-	X
	In situ test parameters	?	?	?

Items indicated as "?" are sometimes used instead of, or in addition to other parameters indicated.

Classification of soil and rock is important for design for several reasons. Soil and rock classifications are often a primary source of information for interpreting stratigraphy, and for evaluating the consistency of conditions from one area of a site to another. Soil and rock classifications also often dictate specific methods that should be used for design, and the associated design parameters that should be characterized. Additionally, soil and rock classification information is often useful for identifying potential concerns that must be considered during design. For example, classification of a soil as high plasticity clay suggests the potential for substantial shrink-swell behavior that should be considered in design.

Finally, site characterization programs should be sufficient to establish reliable values for specific design parameters in each identified stratum. The specific design parameters needed will generally depend on the type of soil/rock present as well as the specific design method adopted for different materials. Design parameters for some materials and methods can be derived from laboratory tests on samples collected by boring and sampling. Design parameters for other methods, particularly those developed for materials

that are difficult to sample (e.g., clean sands, highly weathered or fractured rock), are often based on in situ test measurements such as the cone penetration test (CPT) and the standard penetration test (SPT). Additional guidance regarding interpretation of specific design parameters is provided in Section 2.5 and in GEC 5 (Loehr, et al., 2017).

An important consideration for site characterization activities for drilled shafts is that many drilled shaft design methods are empirical methods that relate drilled shaft performance to some specific form of measurement (e.g. undrained shear strength, CPT tip resistance, etc.). Because the methods are empirical, it is important that design parameters be established using measurements that are consistent with the empirical design relations. Use of site characterization methods that are inconsistent with the empirical methods can have unintended and undesirable consequences. For example, commonly used methods for predicting unit side resistance in clays relate unit side resistance to undrained shear strength, which is known to be subject to varying degrees of bias as a result of sample disturbance. Many common empirical methods are developed using "typical" boring and sampling techniques (often from unconsolidated-undrained triaxial tests on 3-inch diameter Shelby tube samples). These empirical methods therefore inherently incorporate effects of "typical" sample disturbance. If alternative methods are used to characterize undrained shear strength for design (e.g., using the SHANSEP approach that greatly reduces effects of disturbance), one might remove an inherent bias in the empirical design method and actually produce designs that are less reliable than desired despite the fact that more accurate interpretations of undrained shear strength will be produced. The converse outcome also applies if inferior methods are used to characterize undrained shear strength, in which case designs will tend to have reliability that is greater than desired. It is therefore important that site characterization be performed to produce measurements that are consistent with those used to develop the empirical methods that will be used for design.

2.3 INFORMATION REQUIRED FOR CONSTRUCTION OF DRILLED SHAFTS

Additional information beyond that needed for design is usually required by both contractors and engineers for the purpose of establishing appropriate construction methods, selecting proper tools and equipment, making cost estimates, preparing bid documents, and planning for construction. This aspect of site characterization cannot be overemphasized, considering that: (1) the most frequently cited cause of drilled shaft failure is improper construction; and (2) subsurface conditions represent a significant source of construction claims, change orders, and cost overruns (Boeckmann and Loehr, 2016). It follows that careful attention to acquisition and communication of all pertinent information about subsurface conditions can reduce risks of poor performance and the potential for claims, change orders, and cost overruns. Examples of information required specifically for construction are given in Table 2-2.

Drilled shafts bearing on or socketed into rock pose special challenges for construction (Turner, 2006). Many designers assume the tip of the shaft will bear on relatively sound or intact rock and that measures will be taken during construction to verify that assumption. It is critical for both the designer and contractor to have a common understanding of what constitutes adequate bearing conditions in rock and what measures will be taken to locate the shaft tip at the proper elevation. Exploratory drilling conducted at the shaft location prior to construction should include rock coring to a depth that is sufficient to determine that the rock is not a cobble or boulder ("floater"), to evaluate rock quality, and to evaluate the potential for solution cavities or zones of decomposed rock. Boring logs should include clear indication of the depth to acceptable bedrock. If coring into rock is not done prior to construction, it may be necessary to core the rock within and below the rock socket for each drilled shaft during construction to confirm rock quality. For both cases, it is advisable to establish agreement on two issues prior to construction. First, there must be clearly defined criteria for what constitutes adequate rock quality. The criteria could be based on factors such as core recovery, RQD, rock strength, degree of weathering, or

other parameters that can be objectively determined. Second, there must be a clear understanding regarding how to proceed when coring reveals conditions that do not meet the established criteria. This might involve excavation to greater depth. It then becomes necessary to define the method of payment for additional excavation beyond the anticipated depth.

TABLE 2-2 INFORMATION USED FOR DRILLED SHAFT CONSTRUCTABILITY

Application	Information
Selection of appropriate drilling equipment and tools for excavation	• Presence, size, distribution, and hardness of cobbles and boulders • Obstructions such as old foundations, pipes, construction rubble, trees, etc. • Rate of advancement of exploratory boreholes • Torque and crowd of the drilling machine used for exploration • Tools and methods used for sampling • Characteristics of rock mass (depth, strength, hardness, fracturing, RQD, weathering, etc.)
Selection of appropriate methods and materials for excavation support (dry, casing, slurry, combined)	• Cohesionless soils below water table; grain size distribution for coarse-grained soils, including percentage of fines (to assess suitability of polymer slurry use) • Location of free water or seeps, rate of groundwater inflow, and piezometric levels; proximity of surface water infiltration sources (river, lake, ocean) • Presence of artesian groundwater conditions in confined aquifers • Methods of support used for exploratory borings (drilling mud, casing, other); observations of caving (stand-up time); observations of fluid loss • Hardness, pH, and chloride content of groundwater (for slurry construction) • Environmental restrictions on use and disposal of slurry
Match field inspection (quality assurance) procedures with construction procedures	• Anticipated base conditions and requirements for base cleanout • Anticipated Integrity Testing methods • Potential use of specialized inspection tools (borehole calipers, Shaft Inspection Device (SID), downhole cameras, etc.) • Need for supplemental borings/rock cores during construction

In some regions, the depth to bedrock and weathering of rock can be extremely variable. For example, in some karstic environments the rock surface may be pinnacled and highly variable both laterally and vertically as illustrated in Figure 2-1. When such conditions are encountered, it may not be practical to confidently establish tip elevations for individual drilled shafts prior to construction. In such cases, site characterization activities should strive to clearly identify that highly variable conditions are present, to characterize the anticipated variability to the extent possible, and to clearly communicate that highly variable conditions are present so that designers and constructors are aware of the conditions and can take appropriate measures for design and construction. Additional site characterization, which may include completion of pilot holes prior to excavation of individual drilled shafts, coring and probing beneath the tips of drilled shafts, and visual inspection of shaft excavations prior to placing concrete, should then be required during construction to establish final bearing depths.

It is important to recognize that establishing the suitability of rock for meeting design requirements is not equivalent to defining rock for purposes of excavation and payment. A contractor has a right to be paid for rock excavation regardless of its quality as a bearing material, and pay quantities should not be based on suitability of the rock for an engineering design function.

Inadequate characterization of groundwater conditions is one of the most common and costly sources of construction claims, change orders, and cost overruns (Boeckmann and Loehr, 2016). Careful attention should therefore be paid to characterizing groundwater conditions, especially when conditions involve artesian conditions or high rates of seepage. Artesian conditions and zones of high seepage should be clearly indicated on boring logs and described in geotechnical reporting documents. In rock formations,

water inflow to a bored hole is controlled by seepage along discontinuities. This type of flow can vary significantly over short distances and can be a critical factor in drilled shaft construction. It is also not uncommon to observe high seepage rates in one borehole or drilled shaft excavation and little or no seepage in an adjacent hole just a short distance away. It is, therefore, important to observe and record rates of water inflow to exploratory boreholes in rock, and to communicate those observations to potential contractors for estimating the potential for water inflow during drilled shaft construction. Loss of drilling fluids within exploratory borings, and the depths at which such loss occurs, should also be recorded since such conditions may be indicative of cavities or voids that may lead to loss of slurry or concrete during drilled shaft construction.



Figure 2-1 Example of extremely variable rock formation in karstic region.

Where subsurface contamination is detected or anticipated, special measures may be required to ensure worker safety and safe disposal of contaminated cuttings and drilling fluid. When these factors are known beforehand and made clear to all parties, proper measures can be incorporated into construction plans and appropriate payment provisions can be included. When contamination problems are discovered during construction, costs for addressing safety and disposal issues can be significantly greater, involving schedule impacts as well as increased drilling and disposal costs.

An effective way to obtain essential information on drilled shaft constructability is to install one or more full-sized test excavations, referred to as a "technique shaft" (also as a "method shaft" or "trial shaft") during the design phase or at the start of construction. A technique shaft should be of sufficient depth and diameter to reveal problems and difficulties likely to be encountered by a contractor installing production shafts at the same site. Examples of information that may not be obtained easily from exploratory borings but will be obvious during technique shaft construction include: (a) caving or squeezing soil, especially if wash-boring techniques or rotary drilling with casing are used for site investigation, (b) presence of cobbles or boulders that could easily be missed by a small-diameter boring or be mistaken for a layer of rock, (c) elevation at which water will flow into the excavation and the rate of water inflow, and (d) conditions at the shaft tip and effectiveness of base cleanout methods. If there are questions pertaining to placement of reinforcing cages or concrete, a technique shaft can be carried through these stages of construction as well. Technique shafts can also provide important data for design, for example, the degree of sidewall roughness for shafts in rock. It may also be possible to conduct in-situ tests, take downhole photographs or videos, and verify assumptions about tip conditions, all of which can be important for evaluating design parameters. A technique shaft can also be combined with a design-phase load test, providing a wide range of design and constructability information and reducing uncertainty during final design. During construction, however, it is typical to complete the technique shaft prior to

installing the test shaft so any modifications to means and methods derived from construction of the technique shaft can be applied to the test shaft and reflected in load test results.

The information described above and collected specifically for constructability must be made available to bidders to provide them with a basis for making improved cost estimates, and to avoid potential claims related to the withholding of project information. The same information is also needed by the engineer to forecast potential construction methods and construction problems in order to develop specifications for the project, make cost estimates, and perform risk analysis.

2.4 RECOMMENDED MINIMUM SCOPE FOR GEOTECHNICAL INVESTIGATIONS

The scope of a site characterization program should be determined by the complexity of ground conditions, foundation loading characteristics, size and structural performance criteria for the bridge or other structure, acceptable levels of risk, experience of the agency, constructability considerations, and other factors. The primary source of information concerning loading characteristics and the structure will be the Bridge and Structures Office of the state or local transportation agency. Any preliminary plans developed by the structural engineer should be studied and the geotechnical engineer should coordinate directly with the structural engineer and other project staff, preferably through periodic meetings with the design team.

Some information needed to appropriately scope site characterization activities may only be known following a preliminary study of the site. For this reason, site characterization for drilled shaft projects may be carried out through a phased characterization program as described in GEC 5 (Loehr, et al., 2017). Such phased investigations typically include collection of existing site data and geologic information, preliminary investigations intended to identify and characterize soil/rock type and general stratigraphy, followed by one or more detailed investigations intended to refine preliminary interpretations and establish values for design parameters. When properly planned, phased investigations can provide sufficient and timely subsurface information for each stage of project development while limiting the risk and cost of unnecessary explorations. Phased investigations are also fundamental to design-build projects wherein initial investigations are conducted prior to bidding to sufficiently define conditions for preliminary design, and additional investigations are performed by the design-builder after contract award for final design and construction. In the overall design process for drilled shafts presented in Chapter 8 (see Figure 8-1), collection of existing data and preliminary investigations comprise Step 2: Define Preliminary Project Geotechnical Site Conditions. More detailed site characterization constitutes Step 4: Develop and Execute Subsurface Exploration and Laboratory Testing Program for Feasible Foundation Systems. Additional site exploration could be required during construction in some cases.

Article 10.4 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2017a) includes provisions for site characterization and establishing soil and rock properties for foundation design. Article 10.4 recommends a minimum of one boring per substructure (pier or abutment) at bridge sites where the width of the substructure is 100 ft or less and at least two borings for substructures over 100 ft wide for bridge foundations in general. For drilled shafts, it is often desirable to locate a boring at every drilled shaft, especially when the shafts will be founded on or socketed into rock. In practice, this is not always feasible and factors such as experience, site access, subsurface variability, geology, and the importance of the structure will be considered. For multi-shaft foundations composed of smaller diameter shafts (say 6 feet or less) supporting routine bridges in relatively consistent ground conditions, the minimum requirements provided in the AASHTO specifications are likely sufficient. However, similar multi-shaft foundations with larger diameter shafts or more variable subsurface conditions likely justify greater numbers of borings to characterize the volume of ground that will encompass the multi-shaft foundation. For example, for groups of four or more drilled shafts, one should consider placing borings at the

locations of the four corner shafts in each group. Additional borings within interior of the group should also be considered for larger groups unless ground conditions are known to be particularly uniform. For foundations where a single column is supported on a single shaft, borings should be made at the location of every shaft. Borings should also be made at each shaft location when subsurface conditions exhibit extreme variations over short distances, such as can be encountered in karstic limestone or degradable rock formations.

The scope of design phase investigations may also depend on plans for QA/QC during construction. For example, at sites where depth to rock or the condition of rock is highly variable, it may be prudent to require sufficient design-phase borings to capture the potential variability that will be encountered during construction but to complete additional borings as "probe holes" during construction. In such cases, the final shaft depth can be established during construction based on information obtained from the probe holes, and the number of borings required prior to design may be accordingly reduced. In extreme cases, the Florida Department of Transportation has sometimes even required multiple probe holes within a single, large diameter shaft founded in highly variable rock. Figure 2-2 shows illustrations of extreme variability in hard pinnacled limestone that might warrant multiple probe holes during construction for large diameter shafts.

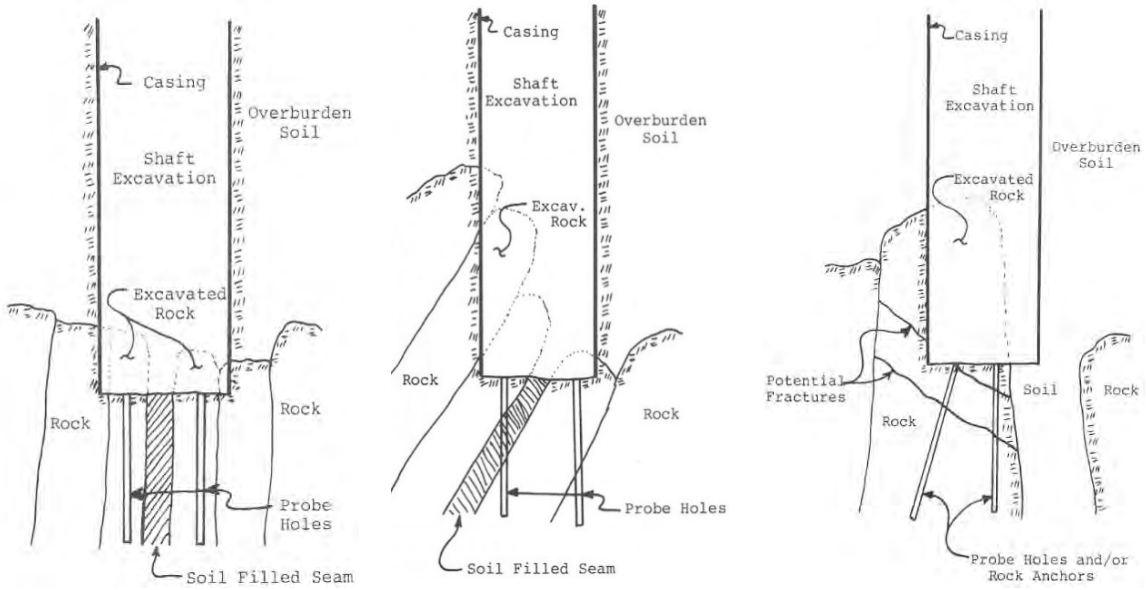


Figure 2-2 Extreme conditions at tip of drilled shafts in hard pinnacled limestone (Brown, 1990).

For secant/tangent pile retaining walls and retaining walls supported on drilled shafts, a minimum of one boring is required for walls up to 100 ft long, and a minimum of two borings is required for walls with lengths of 100 to 200 ft.. For walls greater than 200 ft length, the spacing between borings should be no greater than 200 ft. Additional borings should be considered in front and behind the wall line to define conditions at the toe of the wall and in the zone behind the wall to estimate lateral loads and anchorage capacities, and for global stability analyses. The same considerations identified above for bridge foundations at highly variable sites also apply to drilled shafts for retaining walls.

AASHTO (2017a) recommends the following for depth of borings, for both bridge foundations and retaining walls:

> 'In soil, depth of exploration should extend below the anticipated pile or shaft tip elevation a minimum of 20 ft, or a minimum of two times the maximum pile group dimension, whichever is

deeper. All borings should extend through unsuitable strata such as unconsolidated fill, peat, highly organic materials, soft fine-grained soils, and loose coarse grained soils to reach hard or dense materials'.

'For shafts supported on or extending into rock, a minimum of 10 ft 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. Note that for highly variable bedrock conditions, or in areas where very large boulders are likely, more than 10 ft. of rock core may be required to verify that adequate quality bedrock is present.'

The scope for geotechnical investigations should also consider the need to acquire sufficient numbers of test measurements to reliably establish geotechnical design parameters and assess the variability and uncertainty of those parameters. Guidance regarding interpretation of geotechnical design parameters and the reliability of those parameters is provided in GEC 5.

The above recommendations for borings in soil and rock to be made to two times the maximum group dimension may not be practical or necessary in all cases. For example, if a group of shafts is designed so that tip elevations correspond to the top of rock and the rock mass is known to be competent material, there is no need to extend borings beyond three shaft diameters into the rock. In rock, geologic knowledge based on experience should always take precedence over general guidelines such as those provided here. In rock mass that is known to be uniform and free of cavities, voids, weathered zones, etc., it may not be necessary to drill more than one diameter (10-ft minimum) below the design tip elevation. On the other hand, in highly variable rock mass containing solution cavities, weak zones, boulders in a soil matrix, or other potentially adverse features, borings may need to extend as deep as necessary to verify competent bearing layers. All of the recommendations cited above, for both frequency and depth of borings, are subject to modification based on the level of geologic knowledge of the site and subsurface variability. In general, the more uniform the subsurface conditions and the more experience the geotechnical engineer has with those conditions, the fewer borings are required. For sites with highly varying geologic conditions, where there is little prior experience, a greater number of borings and/or deeper borings may be warranted.

2.5 INTERPRETATION OF GEOTECHNICAL DESIGN PARAMETERS AND REPORTING

Values for geotechnical design parameters should generally be determined according to guidance provided in GEC 5 (Loehr, et al., 2017). When used with AASHTO LRFD methods (or other similarly established design methods), design parameters should generally be selected to represent the mean value of the parameter within a stratigraphic unit. Additionally, the uncertainty in design parameters should be evaluated to ensure that design parameters have appropriate reliability for use with the LRFD methods. In situations where insufficient measurements are available to produce reliable estimates of the mean value for the parameter, a conservative estimate for the parameter should instead be used, as described in GEC 5.

When selecting values for design parameters derived from in situ test measurements, care should be taken to establish whether the in situ measurements are being used as “direct” or “indirect” measurements as described in GEC 5. If the in situ test measurements are used to infer values for common engineering properties (e.g., strength parameters, stiffness, etc.) that serve as the basic inputs for the design method, the in situ measurements should be interpreted as being indirect measurements. Conversely, when in situ test measurements serve as the primary inputs to the design method (e.g., a method that directly relates

unit side resistance to CPT tip resistance), the in situ test measurements should be interpreted as direct measurements as described in GEC 5.

Geotechnical reports developed for sites where drilled shafts will be used should generally be prepared according to guidance in GEC 5 and Federal Highway Administration GEC 14 (Sheahan, et al., 2016). These reports should document the available subsurface information and the interpretation of these data for design purposes. In addition, as described previously in this chapter, the content of reports addressing constructability can have a dramatic impact on bid prices and the potential for claims, change orders, and cost overruns. Careful attention should therefore be paid to making sure that constructability is addressed in geotechnical reporting documents and that those documents are made available to potential bidders.

2.6 DIFFERING SITE CONDITIONS

A common source of contractor claims on drilled shaft construction projects is "differing site conditions" (DSC). Federal law requires that a DSC clause be incorporated into all Federal-Aid Highway Projects. Drilled shaft construction involves inherent risk of encountering conditions differing from those anticipated due to the complexity and variability of natural earth and rock formations and materials. The purpose of the DSC clause is to provide contractors with legal grounds for recovering costs to which they are rightfully entitled when conditions are encountered that differ materially from what a contractor could reasonably anticipate based on the documents available at the time of bidding. Inclusion of a DSC clause is also intended to induce contractors to limit contingencies in their bids, thus promoting lower initial pricing. The best approach for reducing DSC claims is to conduct a thorough site investigation and to disclose all relevant information to contractors bidding on the project.

Geotechnical Engineering Notebook Issuance GT-15 (FHWA, 1996) was prepared to provide guidance to design and construction engineers on the topic of geotechnical differing site conditions. This guideline provides information on adequate site investigation, disclosure and presentation of subsurface information by highway agencies, and the use of such information in mitigating or resolving contractor claims of differing site conditions. Recommendations are provided for disclosure of factual, qualified, and interpretive geotechnical information. A major point made in GT-15 is that the best way to reduce the risk of geotechnical construction problems is early recognition of geotechnical problems during the design stage and designing accordingly. This normally requires conducting an adequate subsurface investigation in advance of final design.

Complete disclosure of all available subsurface information in contract documents is an important factor in both preventing contractor claims and obtaining fair bids for the work to be performed. Subsurface information may be presented in detail in either the contract documents or made available at a central location for bidder inspection. The amount of subsurface information actually presented and the method of presentation in the contract documents can vary depending on the complexity of the project.

2.7 GEOMATERIALS REQUIRING SPECIAL CONSIDERATION

Some geologic environments pose unique challenges for determining material properties for design of drilled shafts or for drilled shaft construction. Examples include:

- Argillaceous sedimentary rock
- Limestone and other carbonate rocks
- Glacial till

- Piedmont residual soils
- Cemented soils

Experience has demonstrated that the geomaterials listed above may require methods adapted to the specific geologic environment. Suggestions on how to approach characterization of engineering properties for drilled shaft design or construction in these materials are presented in Appendix B. Appendix B also provides detailed descriptions of approaches used successfully by state transportation agencies to design drilled shafts in these challenging geomaterials. Application of the term 'special' does not imply that the materials listed above are encountered infrequently. In some locations, these are the predominant geomaterials in which drilled shafts are used, and can provide excellent support. Rather, the term "special" suggests that procedures for establishing engineering properties may require techniques adapted to the unique material characteristics.

2.8 SUMMARY

This chapter provides guidance for developing and executing site characterization programs for projects that include drilled shaft foundations. The guidance provided in this chapter is intended to be used in conjunction with guidance provided in the AASHTO Subsurface Investigations Manual (AASHTO, 1988), Geotechnical Engineering Circular No. 5 (Loehr, et al., 2017), and Geotechnical Engineering Circular No. 14 (Sheahan, et al., 2016). In the chapter, information requirements for design and construction of drilled shaft foundations are described along with guidance for developing appropriate scopes for site characterization investigations. The importance of differing site conditions (DSC) clauses is also discussed. Finally, several geomaterials that require special consideration for site characterization and design are identified and discussed.

CHAPTER 3
GENERAL CONSTRUCTION METHODS

3.1 INTRODUCTION

The effective use and design of drilled shafts requires a knowledge of drilled shaft construction methods, and an understanding that the construction techniques, magnitude of effort and related cost for drilled shaft installation are closely tied to the existing ground conditions at a project site. For successful performance of the completed drilled shafts, the construction technique used must preserve the integrity of the bearing materials and ensure the structural integrity of the cast-in-place reinforced concrete foundation.

In principle, construction of drilled shafts is a very simple matter and follows these basic steps:

1. Excavate the hole, maintaining stability,
2. Clean the hole and prepare for concrete,
3. Place the reinforcement and the concrete, and
4. Finish the top of the drilled shaft for the connection to the rest of the structure.

These steps are simple, but each present a unique set of challenges due to the many variations in ground conditions and design requirements. This chapter provides an overview of the typical methods used to complete the steps outlined above, with more detailed descriptions of the many variations of techniques provided in the following chapters.

In normal contracting practice for transportation projects in the U.S., it is the contractor's responsibility to choose an appropriate method for installing drilled shafts at a given site, and details of the general methods used for construction on a given project can vary with project-specific ground conditions as well as the capabilities, experience, and equipment of an individual constructor. The efficiency and cost-effectiveness of the means and methods chosen for specific circumstances are the contractor's burden, subject to the constraints of the design. Unwarranted interference affecting the contractor's ability to prosecute the work can lead to claims and additional costs.

A good understanding of construction methods is necessary to:

1. identify conditions suitable for drilled shaft foundations, and to estimate costs and schedule,
2. develop constructible designs,
3. execute a site investigation plan to document geotechnical conditions that are important for drilled shaft construction, as discussed in Chapter 2,
4. understand limitations which may be imposed on construction because of potential impacts on foundation performance or nearby structures,
5. develop specifications and inspection procedures that are suitable for the conditions, as discussed in Chapters 14 and 15,
6. develop contract provisions that are appropriate for payment for the work and include appropriate provisions to handle subsurface risks inherent in the work, as discussed in Chapter 14.

For these reasons, designers and other project professionals must be familiar with construction methods. A constructability review by knowledgeable construction professionals during the design phase of the project can often be helpful in improving constructability and cost effectiveness of a design as well as

mitigating potential impacts of construction risks. Drilled shaft construction planning always starts with a thorough understanding of subsurface conditions (as discussed in Chapter 2).

The most important considerations for construction are the ground conditions and the techniques that will be required to maintain stability of the drilled shaft excavation. A stable excavation is necessary to avoid loosening the ground or even collapse of the excavation, which could undermine stability of the worksite and impair the performance of the completed foundation. The size and depth of the hole, the soil or rock to be excavated, and the groundwater conditions all affect the equipment and tooling to be used for the project.

3.2 OVERVIEW OF GENERAL CONSTRUCTION OF A DRILLED SHAFT IN DRY STABLE GROUND CONDITIONS

Under some circumstances, the ground conditions permit an open hole to be excavated without external support. Stable conditions may exist when an excavation is made in strong clays (Figure 3-1) or cemented soils where the groundwater is low and/or the soils have very low permeability. Where ground conditions permit open hole construction techniques to be employed, drilled shafts can be built very quickly and economically. An example is provided by the recent construction of a very large manufacturing facility in central Alabama. At this site 3-ft diameter drilled shafts were installed to a depth of about 45 ft through stiff clay soils into the Selma Chalk formation, a cemented calcareous material with a compressive strength of around 100 to 300 psi. The contractor was able to install 15 to 20 drilled shafts each day, providing a very cost-effective foundation solution for the owner. Another example is the T-Rex project in the Denver, Colorado area, a major interstate project where many areas within this project were suitable for dry open-hole construction of drilled shaft foundations for bridges and retaining walls.



Figure 3-1 Excavation of a Drilled Shaft in Naturally Stable Soils

As the excavation advances, it is important that records (usually in the form of a construction log) be maintained of the soil and rock materials that are encountered, with notes of any obstructions or conditions that might differ from expectations. Where elevations of founding strata vary, the as-built length of the drilled shaft may be adjusted as needed. This log serves as a valuable record of the as-built foundation to confirm that the design intent has been accomplished, and the records often document contract pay items.



Figure 3-2 Visual Inspection of Excavated Materials for Construction Records

Note that the dry stable hole in Figure 3-2 includes a short surface casing which is commonly employed for stability near the ground surface, a guide for the tools, and may serve as a safety barrier if left high enough (Figure 3-2 has a separate safety barrier).

With a dry stable hole, the placement of reinforcement and concrete is relatively straightforward. After any loose material is cleaned from the base of the excavation, the reinforcing cage is lifted and placed into position (Figure 3-3). Consideration of the need for pre-assembly and lifting is required in the design and fabrication of the reinforcement. Concrete used for drilled shaft construction is a special mix with high workability (typically 6 to 8 inch slump for use in a dry hole) which is designed to readily flow through the reinforcement and self-compact with no external vibration. In a dry hole, the concrete can be placed by free-fall techniques provided that a centering device is used to direct the concrete down through the center of the reinforcement to prevent concrete from hitting the rebar or sides of the hole (Figure 3-4).

Completion of the drilled shaft includes some finish work at the top to prepare the connection to the structure. There is typically some contaminated concrete at the top to be removed; even with dry placement of concrete some laitance due to bleed water is normal. In some cases the reinforcement is left extending from the top of the completed drilled shaft and then the splice to the column or footing cage is formed above the top of shaft. For a transmission tower or sign foundation, as shown in Figure 3-5, a pattern of anchor bolts may form this connection, and these may sometimes be installed at the end of concrete placement while the drilled shaft concrete is still fluid. In other cases a short permanent casing or form is left in place so that the splice to the structure is formed below grade.

Where the ground is not inherently stable, drilling fluids and/or casing may be used to support the excavation during construction. The following sections provide an overview of these techniques and how they are used to maintain stability of the excavation during drilled shaft construction. A more detailed discussion is provided in Chapter 5.



Figure 3-3 Lifting and Placing the Reinforcement



Figure 3-4 Free-fall Concrete Placement in a Dry Hole



Figure 3-5 Finishing the Top Includes the Connection to the Structure

3.3 USE OF CASING IN DRILLED SHAFT CONSTRUCTION

A steel casing may be installed to provide support against caving or collapse of the excavation, especially if or when more aggressive excavation tools are employed to advance the excavation into rock or hard materials. The casing can be used to provide temporary support during construction, in which case it is extracted as concrete placement occurs, or it can be installed as a permanent part of the foundation when the drilled shaft must extend through open water or extremely soft and unstable soil layers.

Unless the bearing formation into which the casing is sealed is stable and dry, it will not be possible to use the casing method alone without the addition of drilling fluid.

Installation of casing is generally accomplished in one of three ways.

1. Excavate an oversized hole using the dry method, then place the casing into the hole. This method is suitable only for construction in soils that are generally dry or have slow seepage and that will remain stable for the period of time required to advance the hole to the more stable bearing stratum. The work shown in Figure 3-6 includes excavation through overburden soils to rock in the dry, and then a casing was placed through the soils to rock to prevent caving while the rock socket was drilled.
2. Excavate an oversized hole through the shallow permeable strata using a drilling fluid, then place and advance the casing into the bearing stratum. After the casing is sealed into the underlying more stable stratum, the drilling fluid can be removed from inside the casing and the hole advanced to the final tip elevation by dry methods. A schematic diagram of this approach is provided in Figure 3-7. Note that since the drilling fluid must be flushed out later by the fluid concrete, it must meet all of the requirements for slurry used in the wet method described in Section 3.5.
3. Advance the casing through the shallow permeable strata and into the bearing formation ahead of the shaft excavation, and then excavate within the casing in the dry. With this approach, casing may be driven using impact or vibratory hammers or using a casing oscillator or rotator with sufficient torque and downward force to advance the casing through the soil ahead of the excavation. Even larger upward force may be required to pull the casing during concrete placement. A schematic diagram of this approach is provided in Figure 3-8.

Most steel casing is recovered as the concrete is being placed. In some circumstances, permanent casing may be used and left in place as a form or as a structural element required in the design of the drilled shaft. Instances requiring the use of permanent casing are discussed in Chapter 5, as are other characteristics of temporary and permanent casings.



Figure 3-6 Drilling into Rock through a Cased Hole

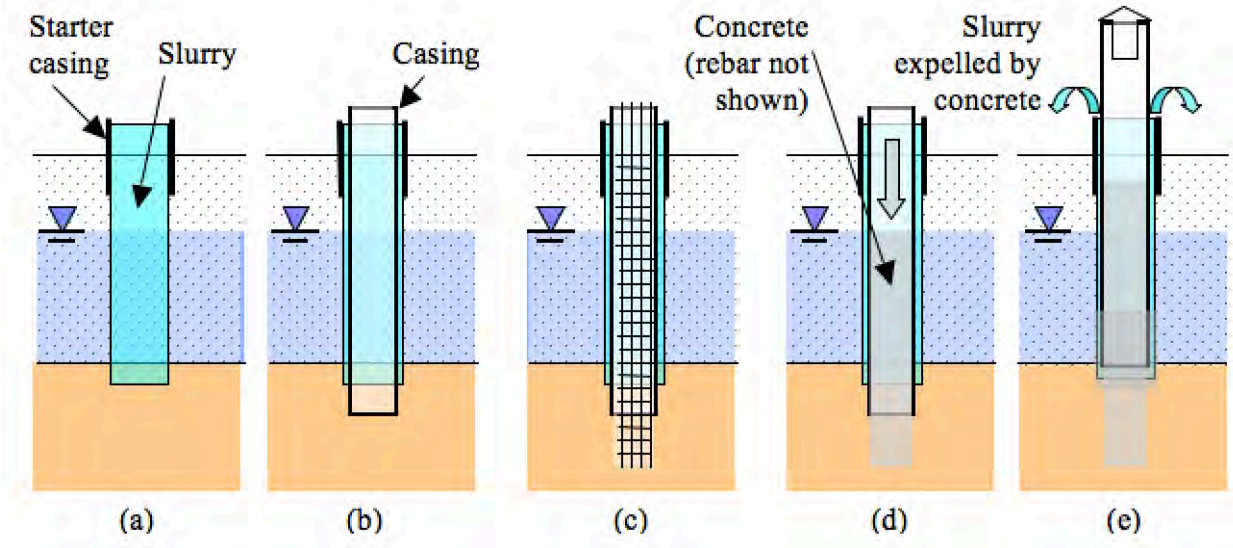


Figure 3-7 Construction Using Casing Through Slurry-Filled Starter Hole: (a) drill with slurry; (b) set casing and bail slurry; (c) complete and clean excavation, set reinforcing; (d) place concrete to head greater than external water pressure; (e) pull casing while adding concrete

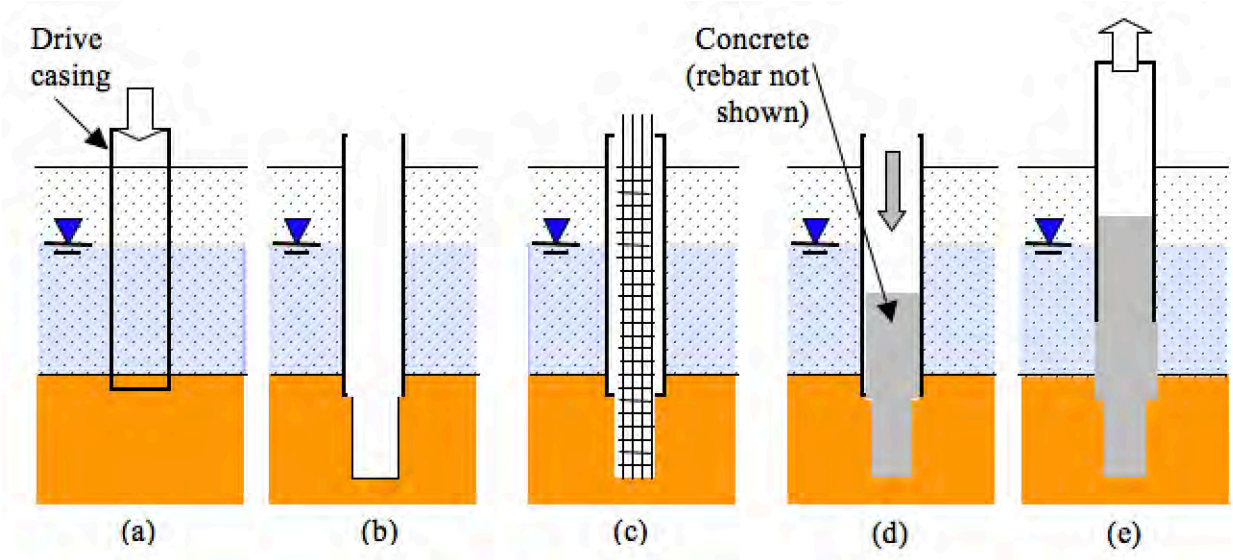


Figure 3-8 Construction Using Casing Advanced Ahead of Excavation: (a) drive casing into bearing stratum; (b) drill through casing; (c) complete and clean hole, set reinforcing; (d) place concrete to head greater than external water pressure; (e) pull casing while adding concrete

In most cases, the shaft excavation will be advanced below the base of the casing for some distance into the bearing formation of soil or rock, using the casing to achieve a seal into this bearing formation to control caving or seepage around the bottom of the casing. Casings may be equipped with cutting teeth to help penetration into a hard layer, but it may not be possible to completely seal the casing, especially if the underlying layer is a rock formation with an irregular or highly fractured surface. In this circumstance a drilling fluid and wet construction (described in the next section) must be employed for the remainder of the work.

If the casing cannot be sealed into a watertight formation, continued attempts to dewater the hole can result in the inflow of groundwater and soil around the bottom of the casing, forming a cavity around the casing. Such a cavity could produce soil loosening, ground subsidence or even collapse at the ground surface. Besides the obvious deleterious effect of ground disturbance and the safety hazard that would be associated with ground movements, this unstable condition can affect adjacent structures. In addition, a large cavity outside the shaft excavation could require a large and unexpected volume of concrete during concrete placement. If a large volume of concrete is lost into a cavity at the time the casing is pulled (step *(e)* in Figure 3-7 or Figure 3-8), the level of concrete inside the casing could drop so much that the seal of the casing into the concrete could be breached allowing inflow of groundwater or drilling fluid. This breach would result in contamination of the concrete, as illustrated in Figure 3-9. To minimize the risk of a large drop in the concrete head within the casing, most contractors would only pull the casing a small amount to break the seal and initiate the flow of concrete behind the casing, and then immediately add more concrete into the casing.

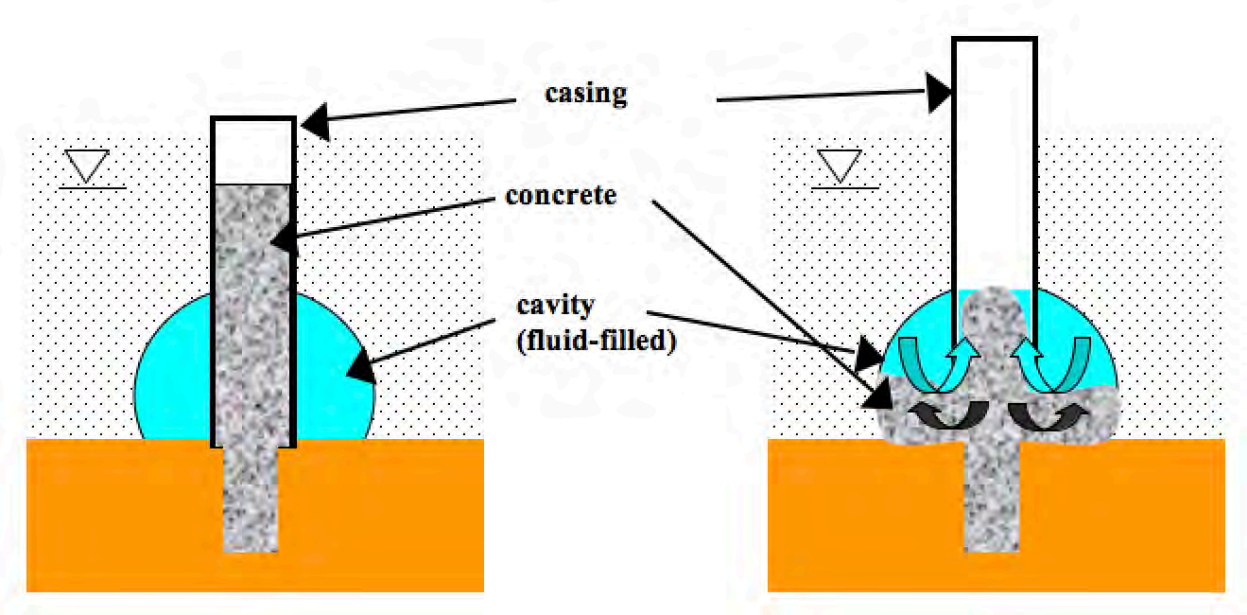


Figure 3-9 Breach of Casing/Concrete Seal During Casing Extraction Due to Cavity

The reinforcing cage is subject to additional constructability requirements for construction with temporary casing. Out-hook bars should be avoided because of the need to withdraw casing over the cage. Since it is generally necessary to release the cage during withdrawal of the casing, it is necessary that the cage be sufficiently stable that it can stand freely under self-weight in the hole during construction without racking or distortion. Because the concrete must flow through the cage to fill the space of and around the casing, there must be sufficient space between bars to permit the free flow of concrete during concrete placement. Additional details relating to reinforcement are provided in Chapter 6.

The concrete used with the casing method must have good flow characteristics in order to flow easily through the reinforcing to fill the space outside the casing and displace any water or slurry around the casing from the bottom up. It is critical that the concrete maintain a hydrostatic pressure greater than that of the fluid external to the casing (trapped slurry or groundwater) as described previously and illustrated in Figure 3-7 and Figure 3-8. The concrete will then flow down around the base of the casing to displace any trapped fluid upward and fill the annular space. The casing should be pulled slowly in order to avoid excessive drag forces from the downward-moving concrete on the rebar cage.

The concrete must also retain workability *beyond* the duration of the concrete placement operations until the casing is completely removed. If the workability of the concrete (slump) is too low, arching of the concrete will occur and the concrete will move up with the casing, creating a gap into which slurry, groundwater, or soil can enter. Typically, the concrete slump should be between 8 and 10 inches for drilled shaft construction requiring casing removal. The rebar cage could also be pulled up along with the casing and stiff concrete. Even if arching within the casing does not occur, concrete with inadequate workability will not easily flow through the cage to fill the space between reinforcing and the sides of the hole. Downward movement of the cage upon casing withdrawal could indicate that the concrete is pulling the cage laterally toward a void or that the cage is dragged downward into a distorted position due to the downdrag from concrete with inadequate workability. Downward movement of the column of concrete will cause a downward force on the rebar cage; the magnitude of the downward force will depend on the shearing resistance of the fresh concrete and on the area of the elements of the rebar cage. The rebar cage can fail at this point by torsional buckling, by slipping at joints, and possibly by single-bar buckling (Reese and O'Neill, 1995). Additional details relating to concrete are provided in Chapter 7.

The casing method of construction dictates that the diameter of the portion of the drilled shaft below the casing will be slightly smaller than the inside diameter of the casing. Most casing is dimensioned by its outside diameter and comes in 6-inch nominal increments of diameter. A contractor would ordinarily use a casing with the increment of outside diameter that is the smallest value in excess of the specified diameter of the borehole below the casing. If the casing diameter is specified in other than standard sizes, special pipe may have to be purchased by the contractor, significantly increasing the cost of the job.

3.4 USE OF DRILLING FLUIDS IN DRILLED SHAFT CONSTRUCTION

Fluids help to provide stability by counteracting the head pressure of the groundwater that would tend to flow into the hole, and by providing some support via the differential head pressure when the fluid head within the hole is greater than the groundwater pressure in the soil. Fluids can include water or water mixed with additives such as minerals (bentonite) or polymer. The additives are used to help contain the fluids within the hole and minimize fluid loss due to seepage out through the borehole wall, thereby allowing the positive head pressure to be maintained. Water mixed with additives to alter the fluid properties is typically called "slurry" and the construction technique is sometimes referred to as "slurry drilling".

The steps in the process of constructing a drilled shaft using drilling fluids for stability are summarized as follows:

1. Excavate the hole while maintaining a positive fluid head pressure at all times
2. Clean the hole and prepare for concrete by removing any loose debris from the base of the excavation and by cleaning the fluids to remove excessive suspended materials
3. Inspect the excavation to ensure that the base is sound and the fluid is reasonably clean
4. Place the reinforcement
5. Place the concrete using a tremie, minimizing the exposure of the concrete to the drilling fluid by maintaining embedment of the tremie below the rising surface of fresh concrete
6. Extract any temporary casing as necessary and clean the top of the concrete surface in preparation for the connection to the structure

Drilled shafts can sometimes be constructed in a wet, but otherwise stable excavation through pervious rock or other strong and stable materials. In such a case, the excavation may simply be filled with water to counter the tendency for seepage into the excavation to occur. An example would be a drilled shaft that has casing seated into rock which is not sufficiently water-tight to prevent seepage into the hole. Seepage into the hole is generally undesirable because it may wash fines into the hole and cause voids around the bottom of the casing which could result in ground subsidence or loss of support around the casing. Seepage into the hole during concrete placement can adversely affect the quality of the concrete. In these situations, water should be added to the excavation to counter this seepage and protect the integrity of the fluid concrete mix. As a general guide, seepage into an excavation which exceeds more than one inch in 5 minutes is considered excessive and the hole should be flooded prior to concrete placement using the tremie method.

To maintain a positive fluid head within the drilled shaft requires knowledge of the groundwater levels in the soil, including the levels in all permeable strata (which may contain groundwater with differing head levels). It is therefore essential for the geotechnical investigation to define the existing groundwater conditions. Where seasonal or other variations in groundwater can be significant, it may be necessary to update the groundwater information at the time of construction.

Where slurry is used to stabilize a drilled shaft excavation, it is necessary that a positive fluid head be maintained at all times within the hole so that a stabilizing internal pressure in excess of the external groundwater head is provided. An excess head pressure of 5 to 10 ft or more within the hole (above that of the natural groundwater) may be sufficient to provide stability through granular soils, if the head differential can be continuously maintained. However, slurry with a higher viscosity than water is typically used to maintain the positive head during excavation because water flows out too fast through permeable strata. It is also important that the drilling operations and tools be used in such a way that pressure reduction ("swab pressure" or suction pressure) below the tool does not cause loss of positive head in the hole. A surface casing is typically used to contain slurry and allow the hole to be filled to the proper level, as illustrated in Figure 3-10.

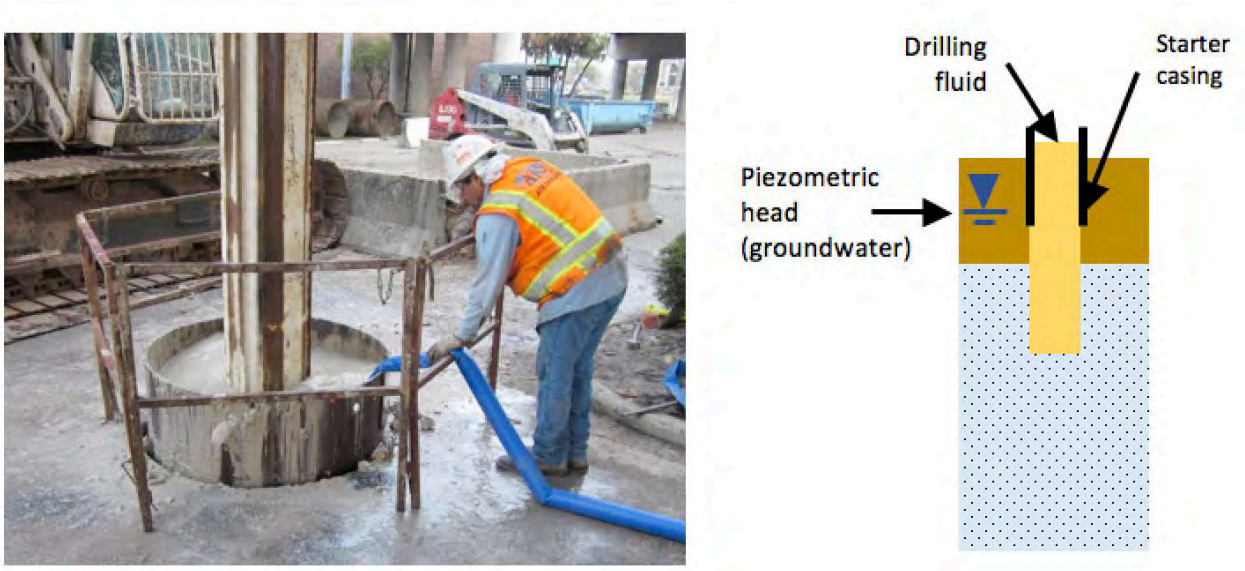


Figure 3-10 Maintaining Positive Fluid Head Pressure in the Drilled Shaft Excavation

The need to maintain positive fluid head can be a challenge and influence the construction methods when groundwater is very shallow. As the excavation advances and the volume of the hole increases, it is necessary to continuously add slurry to maintain the positive head. The drop in slurry level that occurs during extraction of the drill tool can also cause the slurry head to drop below the required minimum level. To address these conditions, an oversized second surface casing has sometimes been used to provide a reservoir for extra fluid volume, as shown in Figure 3-11. Another technique would be to elevate the casing above the ground surface to maintain positive head, as shown in Figure 3-12. Artesian groundwater (levels above the ground surface) can pose special problems to construction because of the need to maintain the slurry level some height above the ground surface to achieve positive head pressure.



Figure 3-11 Use of Surface Reservoir for Drilling Fluid



Figure 3-12 Elevated Surface Casing to Provide Fluid Head Above Artesian Groundwater

As noted above, drilling fluids can include water or a slurry composed of water mixed with minerals (typically bentonite clay) or synthetic polymers. These types of drilling fluids, and their applications, are further described below.

Water as a Drilling Fluid. Water alone can function as a drilling fluid in situations where the formation is inherently stable but permeable, such as some rock formations as noted earlier. Casing may be employed through the unstable overburden to provide a stable environment to excavate the rock using only water. In this situation, it is desirable to maintain a fluid head of water sufficient to avoid infiltration in the rock socket and particularly around the bottom of the casing if a complete seal cannot be achieved into impermeable rock.

Mineral Drilling Fluids. Bentonite clay is the most commonly used mineral additive for drilling fluid, and is widely used in oilfield drilling applications. Bentonite is a clay composed primarily of montmorillonite clay minerals which can absorb water to many times its own weight. When added to water, relatively small amounts of bentonite form a colloidal mixture (referred to as a bentonite slurry)

with the effect of increasing the viscosity of the fluid over that of water, along with a small increase in unit weight.

The other significant property of a bentonite slurry is that some of the minerals are filtered out at the borehole wall as the fluid passes into the soil, thereby forming a "filter cake" that reduces the permeability and helps to contain the fluid. This filter cake formation is the main difference between the performance of bentonite slurry and other commonly used drilling fluids in the construction industry. The filter cake greatly improves the ability of the fluid to maintain stability of the excavation during construction, but can also adversely affect the bond between the concrete and the soil at the interface.

Synthetic (Polymer) Drilling Fluids. In the last 20 years, synthetic (polymer) drilling fluids have replaced bentonite slurry on a majority of drilled shaft applications in North America, although the prevalence varies locally. The use of polymer drilling materials worldwide has also increased dramatically, although perhaps polymers have been more slowly adopted in Europe than in Asia and the Middle East. The type of synthetic polymers used in drilling slurry are long chain-like hydrocarbon molecules which interact with each other, with the soil, and with the water to effectively increase the viscosity of the fluid.

Although there may be some indication of a polymer membrane at the soil interface, there is no formation of a filter cake as with bentonite. As a result of this lack of filter cake, polymers have a greater tendency to lose fluid into the soil around the excavation with time compared to bentonite. The absence of this filter cake also reduces the effectiveness in supporting coarse grained soils and non-circular excavations such as barrettes and slurry wall panels. However, this lack of filter cake appears to provide a benefit in terms of the side resistance at the concrete/soil interface, since the polymers fluids that are in widespread use have not exhibited the detrimental behavior that is associated with bentonite filter cake buildup.

Construction of a Drilled Shaft with Drilling Fluids. The construction of a drilled shaft with drilling fluids is illustrated on the diagram of Figure 3-13. Typical construction would include a starter or surface casing extending above the ground surface as shown in Figure 3-13a. This surface casing may extend as deep as necessary to prevent surface cave-ins and may extend above the ground surface to elevate the surface level of the slurry in the hole, as shown in Figure 3-13b. Note that the groundwater elevation (piezometric surface) is shown by the blue triangle in Figure 3-13. In order to maintain the head of slurry at least 5 ft above the piezometric surface, it is essential that the piezometric surface be known in advance. The presence of overlying cohesive soils may mask the actual elevation of the piezometric surface; drilling into the underlying water-bearing sand stratum without sufficient head of slurry could lead to liquefaction conditions at the base of the hole during drilling, loosening of the stratum, and possibly collapse of the hole or creation of a large cavity.

After completion of the excavation and cleaning of the base, the reinforcing cage is positioned, and then concrete placement is performed using a tremie (Figure 3-13c-e). The tremie delivers concrete to the base of the shaft and displaces slurry upwards. Typically, the slurry is pumped to a holding tank for reuse or disposal. Concrete placement continues through the tremie, always keeping the bottom of the tremie at least 10 ft below the rising surface of the fresh concrete so that the concrete does not mix with the slurry. It is important to avoid potential inclusions of slurry or sediments which may be in suspension within the slurry into the concrete. Therefore, the slurry must be appropriately cleaned of suspended solids to meet the guidelines outlined in Chapter 5. It is also important that the concrete have sufficient workability to flow easily through the tremie and reinforcing throughout the duration of the concrete placement operations, including the time needed to extract temporary casing. Typically, the concrete slump should be between 8 and 10 inches for drilled shaft construction excavated with use of drilling fluids. More details on the important properties of concrete are provided in Chapter 7.

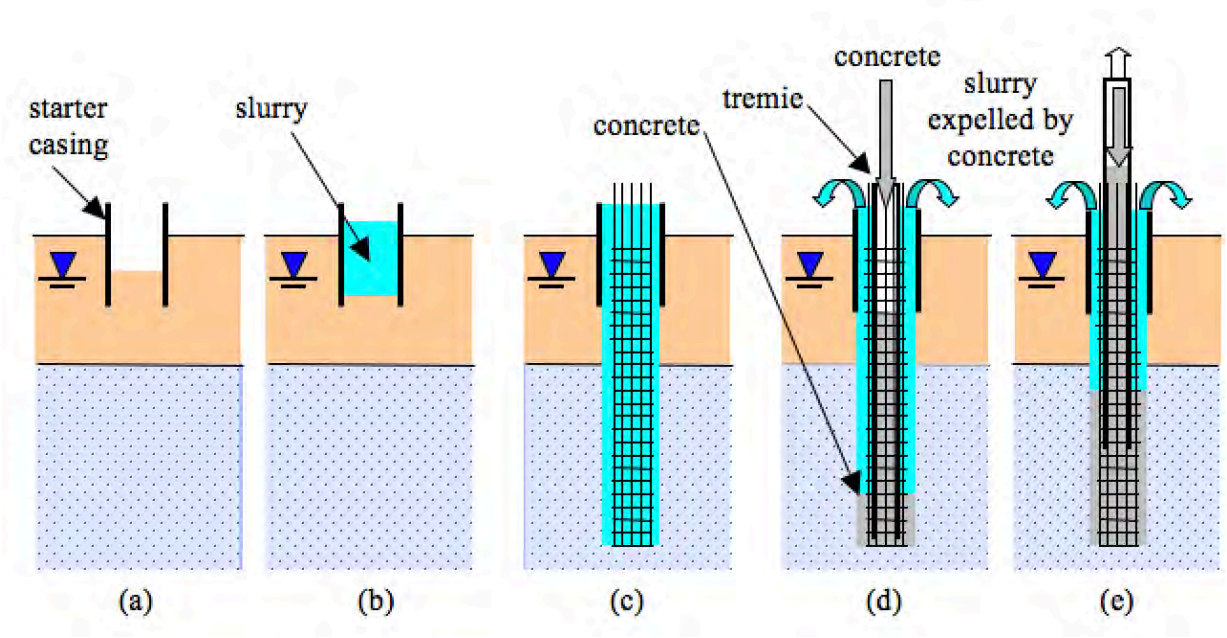


Figure 3-13 Slurry Drilling Process: (a) set starter casing; (b) fill with slurry; (c) complete and clean excavation, set reinforcing; (d) place concrete through tremie; (e) pull tremie while adding concrete

When advancing a shaft excavation through unstable soil using full length temporary casing, drilling fluid head is needed within the casing to avoid an unstable bottom condition as illustrated in Figure 3-14. If excavation within the casing is performed as the casing is advanced, the construction crew should generally maintain a soil plug of sufficient thickness within the casing to avoid bottom heaving. The plug should be maintained until the excavation reaches either a stable stratum (such as rock, cemented soil, or cohesive soil) or the final shaft bottom elevation.

Upon completion of the shaft in cohesionless soil, care must be taken to avoid instability at the base of the casing as bottom heave could produce loosening of the bearing stratum. The drilling fluid inside the casing should be kept at a high elevation by pumping to maintain an excess head in the hole and a positive seepage pressure against the soil at the base. If the casing is seated into strongly cemented soil, very stiff to hard cohesive soils or rock, bottom stability may be less of a concern.

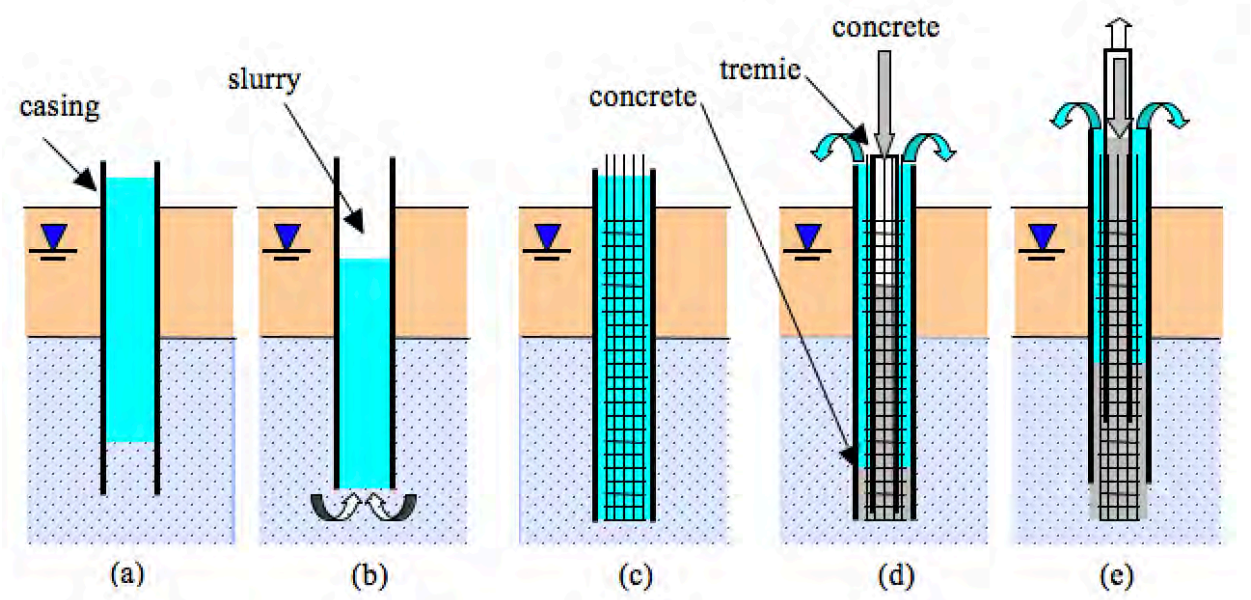


Figure 3-14 Wet Hole Construction Using Full Length Temporary Casing: (a) advance casing and excavating, while maintaining soil plug within casing through caving soils; (b) bottom instability due to inadequate slurry level and/or soil plug at base in caving soils; (c) complete and clean excavation, set reinforcing; (d) place concrete through tremie; (e) pull tremie, casing while adding concrete

3.5 THE ROLE OF ARCHING IN EXCAVATION STABILITY

Arching is an important concept for understanding how the techniques described above can be effectively employed for drilled shaft excavation and stability. When a circular vertical hole is excavated into the soil, arching allows the in-situ lateral stresses in the ground to be transferred around the opening so that the opening can be maintained (Figure 3-15a). Even with the use of casing or drilling fluids for support, most of the lateral stresses in the ground must transfer around the hole via arching.

As the drilled shaft excavation is advanced, the soil around the hole moves inward slightly, thereby allowing lateral stress to be transferred via arching around the hole as shown in Figure 3-15a. This transfer of radial stress to tangential stress around the hole allows a small amount of fluid pressure to stabilize the excavation. Arching can also effectively reduce the external earth pressure acting on casing.

Geotechnical engineers can relate the movement required to mobilize arching to earth pressure forces on retaining structures. As a retaining wall moves toward the excavation, the lateral earth pressures reduce from the at-rest condition to an "active" earth pressure condition as the soil strength is mobilized. Engineers typically design earth retaining structures to resist this lower "active" earth pressure force but recognize that some wall movement will be observed. The reduction in lateral stress in the radial direction due to arching around a vertical hole is far more pronounced than for a two dimensional retaining structure. A vertical cut can be sustained only in soil with sufficient cohesion that the active earth pressure force goes to zero.

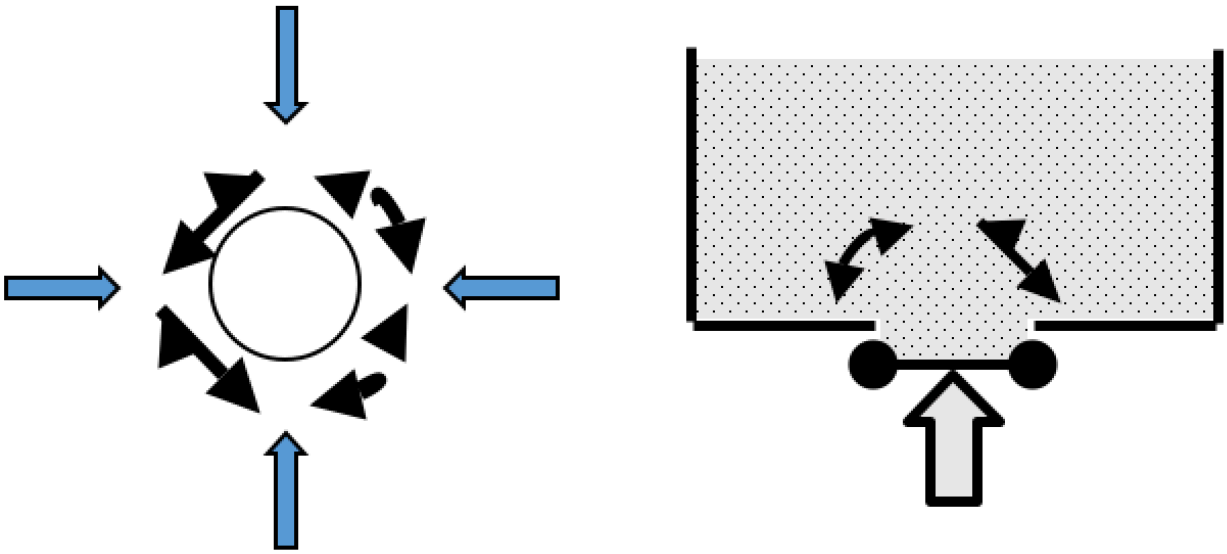


Figure 3-15 a) arching around a hole (plan view) b) schematic of Terzaghi's trap door experiment

This effect was demonstrated in a famous experiment by the father of soil mechanics, Karl Terzaghi, in which he measured the force required to support a trap door in the bottom of a sand-filled box as sketched in Figure 3-15b. His measurements demonstrated that the force on the trap door was greatly diminished as the door moved down away from the box, and explained this reduction as attributed to the arching effect as the interlocking soil particles redistribute the stresses around the opening. The force was observed to diminish to a much smaller magnitude for dense sand than for loose sand, an effect that was recognized as the more effective arching in the stronger soil. Additionally, arching developed at smaller displacements in the stronger soil compared to the loose sand, an observation consistent with the smaller retaining wall movement needed to develop active earth pressure conditions in dense soils. As the door progressively moved further, the arching collapsed and the force on the door returned to levels near the original values.

Similar principles are at work during drilled shaft construction. The stabilizing radial stress provided by 10 ft of excess fluid head pressure is relatively low (about 650 psf) compared to in-situ lateral earth pressures, but it is still observed to be quite effective at maintaining stability. The inward movement of the soil is accommodated during casing installation by using a cutting shoe at the bottom of the casing to reduce friction on the outer face of the casing, but this small overcut also allows some stress relief on the casing. Without this allowance for inward movement around the casing, the lateral stress on the casing would be high and the casing can become stuck. Digging a relief hole ahead of the casing accomplishes a similar stress relief function, allowing arching around the hole to develop and thereby reducing the radial stress on the casing.

Factors affecting the arching stress reduction include:

- The depth of the overburden soil
- The diameter of the hole
- The friction angle of the granular soils
- The relative inward displacement around the hole

Experienced constructors know that denser soil (higher friction angle) is more easily stabilized than loose soils, and the reason for this observation is explained by arching. Looser soils will also require greater

inward displacement to develop arching. A small diameter hole is more forgiving than a larger diameter hole, because arching is more easily accomplished. And just like Terzaghi's trap door experiment, if the radial support is insufficient or too much inward movement occurs, collapse of the soil ensues and the radial stress on the support casing increases.

The science of arching that is underlying these important concepts is well established and applied to other fields of civil engineering, including culvert design and even masonry arch structures that date back thousands of years. The application to drilled shaft construction remains largely an art at the present, due to the many variables and unknowns relative to soil conditions and the dynamics of the drilling process.

3.6 POST-GROUTED DRILLED SHAFTS

For some projects, drilled shafts may be post-grouted to produce improved performance compared to that for a similar ungrouted shaft (FHWA-HIF-17-024 by Loehr, et al., 2017). Post grouting generally entails installing a grout delivery device at the tip of drilled shafts by attaching the device to the bottom of the reinforcing cage, as shown in Figure 3-16, so that it rests on the bottom of the excavation prior to placing concrete. Following concrete placement, and after the concrete has gained sufficient strength, grout is injected through the installed grout delivery device at relatively large pressure using a simple pump. The grout most commonly used for post grouting is a neat cement grout composed of water and cement. The degree of improvement achieved due to post grouting varies with the magnitude of the grout pressure applied, which commonly ranges from 100 psi to over 900 psi, which is typically limited by either the capability of the pumping equipment or the effective overburden pressure in the soil at the depth of the shaft. Post grouting is a subject of ongoing research. Additional information regarding post-grouted drilled shafts can be found in FHWA-HIF-17-024 (Loehr, et al., 2017), and preliminary guidance for application of post grouting for drilled shafts is provided in Appendix F.

Grouting is also sometimes performed on drilled shafts after-the-fact by coring to the tip and stem grouting. This type of grouting is fundamentally different from post grouting in that it is generally performed as a remedial measure to repair drilled shafts determined to have concrete defects. While this is a viable remediation technique, it is a costly and difficult procedure, particularly for deep shafts, and should not be considered as post grouting.



Figure 3-16 Post Grouting Devices Attached to Reinforcing Cage: Flat-jack device (left) and Tube-a manchette device (right)

3.7 SUMMARY

An understanding of drilled shaft construction is critical to a successful foundation design and installation for the reasons outlined in the introduction to this chapter. Although the means and methods of construction are usually delegated to the contractor (along with the contractual obligation to complete the work in a timely manner), engineers must recognize that construction procedures have a major influence on the performance of the drilled shafts. The design methods that are presented in this manual generally do not distinguish among construction methods, but assume that good practices are followed. There may be occasions when it is necessary for the designer to specify a particular construction method; for example, use of full-depth casing to protect adjacent structures, but doing so will almost always add significantly to the cost of the job. Similarly, the specifications may exclude one or more methods that the designer considers unsuitable for the existing ground conditions; for example, precluding the use of the dry method of construction where the risk of caving is considered unacceptable, or requiring the use of permanent casing where extremely soft soils are present.

This chapter provided an overview of general construction methods used for drilled shafts. Subsequent chapters provide more details on the specific issues relating to selection of construction tools and materials.

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CHAPTER 4
TOOLS AND EQUIPMENT

4.1 INTRODUCTION

Most contracts for drilled shaft construction establish that the means and methods are the contractor's responsibility. Therefore, the final choice of the types of drilling rigs and drilling tools that are to be used to make excavations for drilled shafts on a specific project are almost always chosen by the contractor. These choices are made based upon:

- The subsurface conditions that are encountered as a part of the site investigation and presented to bidders via the contract documents. As discussed in Chapters 2 and 3, the geotechnical investigation is essential to appropriately characterize the existing subsurface conditions for the purpose of construction planning.
- Additional indications of subsurface conditions that may be revealed as a part of a site visit by the prospective bidder. Therefore, it is important that the site be accessible to potential contractors. In some cases it may be warranted to perform a pre-bid exploratory shaft excavation so that bidders have an opportunity to directly observe conditions in a full size shaft excavation.
- The contractor's personal experience in similar geologic conditions.
- The contractor's available equipment and experience with that equipment.
- The experiences of other contractors in the local area on similar projects and in similar geology, provided this information is available to the contractor. Where a transportation agency has documented information available on previous drilled shaft projects, this information can be extremely valuable in terms of minimizing uncertainty and contingency costs in the bid, and in avoiding potential claims. There is also great value in post-construction documentation of "lessons learned" for future use by transportation agencies.

The choice of rigs and tools is critical to the success of a project. Sometimes an apparently minor change in a drilling tool can change the rate of excavation dramatically. Because of the importance of selecting proper tools and equipment, it is critical for both engineers and inspectors to understand the general types of rigs and tools available in the United States. Although the burden of risk in the choice of specific tools and equipment is typically the contractor's responsibility, the list above underscores the importance of engineers and owners to understand the types and capabilities of tools and equipment, and the information needed by the contractor to make an informed decision regarding their selection.

This chapter provides a general description of the drilling machines and tools used for excavation of drilled shafts.

4.2 DRILLING MACHINES

The machines used for drilling shaft excavation have evolved over the years from primitive mechanical systems (e.g., Figure 4-1 from the 1930's) supplemented by heavy reliance on manual excavation to sophisticated and powerful hydraulic machines with extensive in-cab instrumentation and controls. This section provides an overview of the broad range of drilling machines available in current (2018) U.S. practice.

4.2.1 Overview of Rotary Systems

Most excavations for drilled shafts in the United States are made by some type of rotary-drilling machine. The machines vary greatly in size and in design, as well as by the type of machine on which the drilling rig is mounted. The machine transmits force from the power unit through the rotary to a kelly bar to the tool attached to the end of the kelly, as illustrated on Figure 4-2.



Figure 4-1 An Early Drilled Shaft Machine and Crew

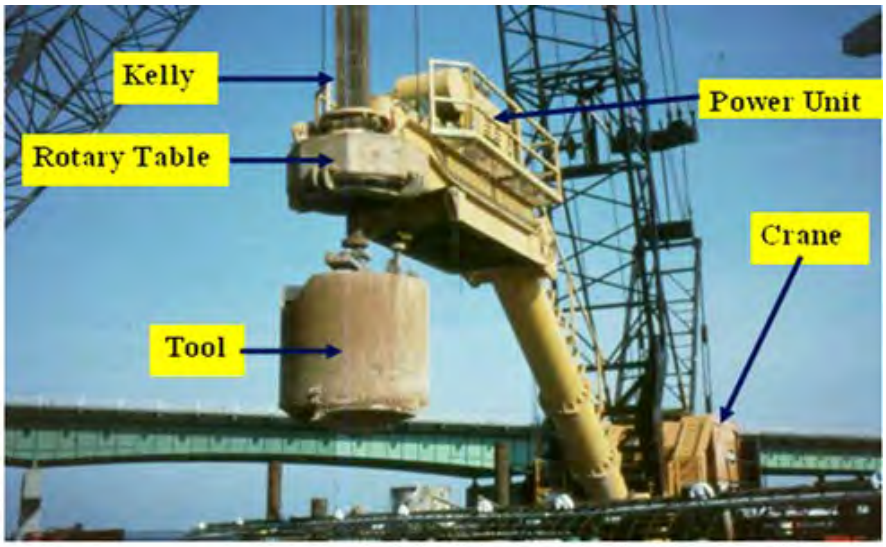


Figure 4-2 Drill Rig Terminology

The capacity of a drilling rig is often expressed in terms of the maximum torque that can be delivered to the drilling tools and the "crowd" or downward force that can be applied. Other factors can have great impact on the efficiency of the rig in making an excavation, particularly the type and details of the drilling tools, but the torque and crowd are important factors affecting the drilling rate.

Torque and crowd are transmitted from the drilling rig to the drilling tool by means of a drive shaft of steel, known as the kelly bar, or simply the "kelly." The drilling tool is mounted on the bottom of the kelly. Kellys are usually either round or square in cross section, and may be composed of a simple single piece (up to about 60 ft long) or may telescope using multiple inner sections to extend the depth to which the kelly can reach. Square kelly bars often require a worker to insert a pin to lock the outer bar to the inner telescoping kelly piece, whereas round kellys often include an internal locking mechanism. In some rigs, the weight of the kelly and the tool provides the crowd. In others, hydraulic or mechanical devices are positioned to add additional downward force during drilling.

Specific details relating to the capabilities of individual drilling machines are readily available on the websites of equipment manufacturers. A contractor will normally provide these details as a part of the drilled shaft installation plan for a specific project.

4.2.2 Mechanical vs. Hydraulic Systems

The drilling machines used in the drilled shaft industry are typically powered by either mechanical or hydraulic systems. Examples of each type are shown in Figure 4-3.

A typical mechanical drive system delivers power to the rotary via a direct mechanical drive shaft or sometimes a right angle drive from a multiple speed transmission. This type of system has a long history of use, is mechanically simple, and is relatively lightweight. Most lightweight truck-mounted drill rigs use direct drive mechanical systems. Large crane attachments, as shown in Figure 4-2, are also direct drive mechanical systems.

Recent years have seen an increase in the use of hydraulic systems to deliver force to the rotary table. The potential high pressures available in modern hydraulic systems can provide rigs with a higher torque range, and the use of the hydraulic drive allows the rotary to move up and down the mast rather than restrained to a location fixed by the drive system. The movable rotary provides versatility in that the rotary can elevate above casing and even be used to install casing. Hydraulic rigs are sometimes heavier and more expensive than a similar size mechanical machine.

Mechanical drive systems often apply crowd through a pull-down system applied to a drive bar atop the kelly, in which the drive bar is guided within the leads as it travels up and down the mast. Although the same crowd system can be applied to a hydraulic driven kelly, more often the crowd in a hydraulic system is applied through the rotary.

4.2.3 Methods of Mounting Drilling Machine

The drilling machine must be mounted on some type of carrier in order to drill and move about the site. The type of carrier has an effect on the versatility of the machine and the efficiency of the overall operation. Drill rigs may be mounted on trucks, crawlers, excavators, cranes, or may even be designed to operate directly as a top drive unit mounted onto a casing. The following sections provide a brief description of drill rigs by methods of mounting the machine.



Figure 4-3 Mechanical (left) and Hydraulic (right) Powered Drilling Machines

4.2.3.1 Truck Mounted Drilling Machines

Mobility is the greatest advantage of truck-mounted drilling machines, which can range widely in size and drilling capabilities. As shown in Figure 4-4, truck-mounted rigs can range from small, extremely mobile rigs most suited to small holes to large, heavy rigs capable of drilling rock. If the site is accessible to rubber-tired vehicles and conditions are favorable for drilling with truck-mounted rigs, construction of drilled shafts can be accomplished very efficiently with these machines. With the mast or derrick stored in a horizontal position, lighter units can move readily along a roadway. The truck can move to location, erect the derrick, activate hydraulic rams to level the rotary table, and begin drilling within a few minutes of reaching the drilled shaft location.

Truck-mounted rigs are normally mechanically-driven with a fixed rotary, and therefore may have limited capability to reach over tall casing or to handle tall drilling tools. The space below the rotary table can be increased by placing the rig on a ramped platform, but this procedure is obviously slow and expensive and would be used only in unusual circumstances. However, some truck-mounts are now supplied with a hydraulic sliding rotary which can overcome many of the limitations of older truck-mounted rigs.

While the truck-mounted unit has a secondary line with some lifting capacity, that capability is necessarily small because of the limited size of the derrick. The drilling tools can be lifted for attachment to and detachment from the kelly, but, if a rebar cage, tremie or casing must be handled, a service crane is usually necessary. Some truck rigs can handle light rebar cages and tremies of limited length.



Figure 4-4 Truck Mounted Drilling Rigs

4.2.3.2 Crane Mounted Drilling Machines

A power unit, rotary table, and kelly can be mounted separately on a crane of the contractor's choice, as shown in Figure 4-5. Crane mounted drill rigs can have substantial capabilities and versatility on a bridge project, especially over water. The crane-mounted machine is obviously less mobile than a truck unit. Mobilization to the jobsite generally requires “rigging” or assembly of the equipment with significant cost and effort.

Power units of various sizes can be utilized to supply large torque at slow rotational speeds to the drilling tool. Usually, the downward force on the tool is due to the dead weight of the drill string, but the dead weight can be increased by use of heavy drill pipe (drill collars), "doughnuts," or a heavy cylinder. Special rigging is available for crane machines that will apply a crowd for drilling in hard rock. The cross-sectional area of the kelly can be increased to accommodate high crowds.

The framework, or "bridge," that is used to support the power unit and rotary table can vary widely. The rotary table may be positioned 75 ft or more from the base of the boom of a crane by using an extended mount. The ability to reach to access the hole from a distance makes crane mounted machines very attractive in marine construction when working from a barge or work trestle. The bridge for the drilling unit can also be constructed in such a way that a tool of almost any height can fit beneath the rotary table. Therefore, crane-mounted units with high bridges can be used to work casing into the ground while drilling, or for accommodating tall drilling tools.

A service crane, or the drilling crane itself, is used on the construction site for handling rebar cages, tremies, concrete buckets and casings. The secondary lift line on the drilling crane can be used for common lifting by tilting the derrick forward and away from the rotary table, thus making the crane-mounted drilling unit a highly versatile tool.



Figure 4-5 Crane Mounted Drilling Rig

4.2.3.3 Crawler Mounted Drilling Machines

Crawler mounted drilling machines may be less mobile than truck mounted equipment for accessible sites, but can provide excellent mobility on the jobsite. Compared to a crane mounted rig, the drilling equipment is usually a permanent fixture on the crawler with a fixed mast serving as the lead for the rotary or kelly guide system. The crawler mount is the most common system used for hydraulic powered rigs, although it is also a popular system for conventional mechanical rigs; both types can be mounted on crawler equipment.

Lightweight crawler mounted drilling machines can be extremely versatile for work on difficult to access sites for applications such as slope stabilization, sound wall foundations, and foundations for signs, towers, or transmission lines. An example of a mobile crawler mounted drill rig is shown in Figure 4-6.

4.2.3.4 Excavator Mounted Drilling Machines

Another type of crawler mount that has advantages for some special applications is the placement of the drilling machine on the arm of an excavator, as shown in Figure 4-7. These rigs are almost always hydraulic, utilizing the hydraulic system common on an excavator. The advantage of such a mount is that the rig can reach a difficult to access location with low headroom or with limited access immediately adjacent to the hole. Low headroom equipment is often advantageous for applications such as a sound wall where utility lines are overhead, or when installing shafts below or very near an existing bridge structure. The use of low headroom equipment has obvious limitations in terms of the depth and size of hole that can be drilled efficiently. Reduced productivity in drilling under low overhead conditions will affect costs.



Figure 4-6 Crawler Mounted Drilling Rig



Figure 4-7 Excavator Mounted Drilling Machines for Restricted Overhead Conditions

4.2.3.5 Oscillator/Rotator Systems

Oscillator and rotator systems are hydraulic-driven tools for advancing and extracting casing. The casing often is a segmental pipe with bolted joints. The oscillator or rotator grips the casing with powerful hydraulic-driven jaws and twists the pipe while other hydraulic cylinders apply upward or downward force. An oscillator twists back and forth, while a rotator (a more expensive machine) can rotate the casing through a full 360° when advancing casing. An example of an oscillator with segmental casing is shown in Figure 4-8. A rotator is shown installing permanent casing into rock in Figure 4-9.

The tremendous twisting force of these powerful machines must be resisted by a reaction system. The oscillator in Figure 4-8 is resisted by an arm extending to a large crane, and this crane uses dead weight to provide friction of the tracks on the pile-supported work trestle extending into the river. The vertical force acting to push the casing down is normally restricted to the dead weight of the casing plus machine, but the vertical force to pull the casing out (which may be much larger, after the casing is embedded into the soil and may be partially or completely filled with concrete) must be resisted by the work trestle system or the bearing capacity of the ground surface if the machine is on land. The axial and torque capacity of the entire reaction system must be carefully designed (normally by the contractor) to be sufficient for the machine to work efficiently.

Excavation within the casing is often made using a clam or hammergrab, although a rotary drilling machine can be mounted on the casing to operate as a top-drive unit. It is also possible to excavate sand within the casing using a dredge pump or airlift system. Care must be used so as not to remove the soil below the casing and, as with any type of circulation drilling (discussed in the following section), fluid must be pumped into the casing sufficiently fast as to maintain a positive head of water. The oscillator/rotator systems are often used with a fully cased hole, although the drilled shaft excavation can be extended into rock or stable soils below the bottom of the casing.



Figure 4-8 Oscillator Machine



Figure 4-9 Rotator Machine

4.2.3.6 Casing Mounted Top-Drive Systems

Casing mounted top-drive systems are used with reverse circulation drilling, since the rotary machine is mounted on the casing itself. The basic principle of reverse circulation drilling uses a full-face rotary cutting head to break up the soil or rock, and an airlift system is used to pump the drilling fluid containing spoil away from the cutting surface. The drilling fluid is then circulated through a desander and/or settling basin, and returned to the shaft excavation. Slurry or water may be used as the drilling fluid, depending upon the stability of the hole and the length of casing. An example of a top-drive system used for the Fore River Bridge in Massachusetts is shown in the photos of Figure 4-10. The photo on the right shows the machine during operation with the circulation system in place; cuttings are lifted from the bottom of the excavation through the central pipe, through the swivel at the top of the drill string, and through the discharge hose to a spoil barge. As the hole is advanced, short sections of drill pipe are added. The photo at left shows the top drive system with the drill being inserted. The cutting head and lowermost portion of the drill string is shown on Figure 4-11.

The top-drive system mounted on the casing must react against the casing during drilling, so the casing must be sufficiently embedded into firm soil to provide a stable platform on which the machine can work. The excavation below the casing must be stable not only for support of the casing, but also to avoid the collapse of soil or rock into the hole above the cutting head that could make the cutting head difficult or impossible to retrieve. The casing may be installed using a vibratory or impact hammer, or using an oscillator/rotator.



Figure 4-10 Top-Drive Reverse Circulation Drill



Figure 4-11 Reverse Circulation Cutting Heads for Top-Drive Drill

4.2.4 Other Excavation Systems

Although the vast majority of drilled shafts are excavated using rotary machines, other systems may be employed to advance an excavation into the subsurface for a wall or foundation. These include manual techniques and excavation using grab tools or slurry wall equipment.

Manual excavation (Figure 4-12), i.e., vertical mining, has been employed for many years and is still a viable technology in some circumstances, such as for underpinning of existing structures. Excavation using workers below ground obviously requires great attention to safety considerations and is usually quite expensive compared with alternatives. Manual excavation is usually only considered where mechanized equipment is ineffective or where the location is inaccessible, such as to remove a boulder or rock or in a confined space where a heavy machine cannot be positioned. For dry excavations into very strong rock, there may be circumstances where hand excavation might sometimes be employed, for example when it is necessary to penetrate steeply sloping rock, as in a formation of pinnacled limestone, where ordinary drilling tools cannot make a purchase into the rock surface.



Figure 4-12 Manual Excavation in Rock

Safety precautions must be strictly enforced when hand mining is employed. The overburden soil must be restrained against collapse, the water table must be lowered if necessary, and fresh air must be circulated to the bottom of the hole.

Other non-rotary excavation techniques may include the use of a grab or clam or hydromill, as in the construction of rectangular diaphragm wall panels. When used as a foundation, an individual panel is often referred to as a "barrette." These panels can be efficiently oriented to resist large horizontal shear and overturning forces in addition to axial loads, and can even be post-grouted to enhance capacity. The use and testing of barrette foundations in Hong Kong is summarized by Ng and Lei (2003). A barrette is typically excavated under mineral slurry to maintain stability of the excavation.

Photos of a clam system are shown in Figure 4-13; these may have a hydraulically controlled guide system to maintain alignment. Photos of a hydromill (or hydrofraise, as it is known in Europe) are illustrated in Figure 4-14. A hydromill or "cutter" is typically used to excavate rock, and cuts the rock with two counter-rotating wheels at the base of the machine. The excavated materials are lifted from the cutting face using an airlift or pump to circulate the slurry similar to the reverse circulation drill described in Section 4.2.3.6.

4.2.5 Summary

This section outlines what may appear to be a dizzying array of choices of machines for excavating a drilled shaft. The variety of machines available to contractors reflects the maturation of the foundation drilling industry and the development of specialized equipment to optimize productivity for particular applications. The range of mounting systems for the drilling machines, and torque and crowd capabilities of modern equipment has extended the size, depth, and potential applications of drilled shaft foundations far beyond those considered feasible a few decades ago. Still, the most common method used to excavate the majority of drilled foundations for transportation structures is that of a simple rotary drilling machine turning a tool at the bottom of a hole and removing soil or rock one auger or bucketful at a time. Although the capabilities of the drilling machine are critical to the ability of the constructor to complete the drilled shaft excavation to the size and depth required, the choice of drilling tools is often as (or more) important to the productivity of the excavation process. Drilling tools are discussed in the following section.

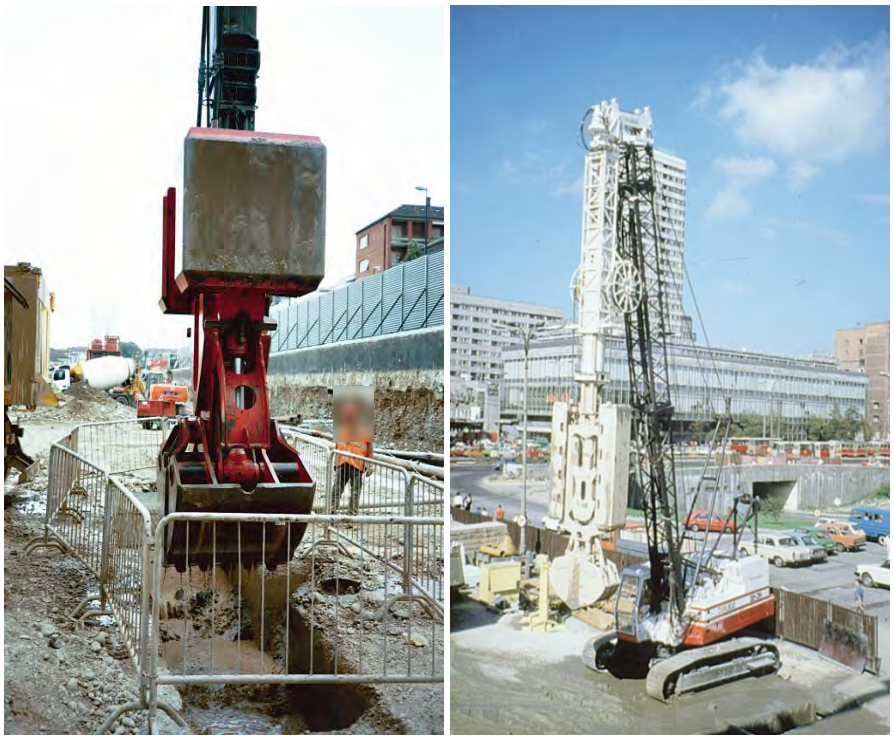


Figure 4-13 Excavation of a Diaphragm Wall or Barrette Using a Clam



Figure 4-14 Excavation of a Diaphragm Wall or Barrette Using a Hydromill

4.3 TOOLS FOR EXCAVATION

4.3.1 Rotary Tools

The tool selected for rotary drilling may be any one of several types, depending on the type and physical properties of soil or rock to be excavated. Rotary tools described in this section include augers, buckets, coring barrels, full faced rotary rock tools, and other specialized rotary tools for drilling soil and rock The tools for rotary drilling are typically available in sizes that vary in 6 inch increments up to approximately 10 ft in diameter. Larger sizes are available for special cases.

Often, small details in the design of a tool can make a huge difference in effectiveness. For example, it is necessary that the lower portion of the tool cut a hole slightly larger than the upper part of the tool to prevent binding and excessive friction. It would not be unusual for one driller to reach refusal with a particular tool while another driller could make good progress with only a slight adjustment to the same tool. Different contractors and drillers will select different tools for a particular task and in many instances will have their own particular way of setting up and operating the tool. Important details in apparently similar tools may vary, and it is not possible to describe all "standard" tools that are in use in the industry.

The following sections give brief descriptions of some of the common tools used in rotary drilling.

4.3.1.1 Augers

This type of drilling tool can be used to drill a hole in a variety of soil and rock types and conditions. It is most effective in soils that have some degree of cohesion, and rock with low to moderate strength and hardness. The auger is equipped with a cutting edge that during rotation breaks the soil or rips the rock, after which the cuttings travel up the flights. The auger is then withdrawn from the hole, bringing the cuttings with it, and emptied by spinning. Difficulties can be encountered when drilling in cohesionless sands where soil slides off the auger flights, and in some cohesive soils where the tool can become clogged.

Augers for drilling soil and rock vary significantly depending upon the type of material to be excavated. The following sections describe various types of augers used in foundation drilling.

4.3.1.1.1 Earth Augers

Earth augers may have a single or double cutting surface, as shown in Figures 4-15 and 4-16, respectively, and many have a central point or "stinger" that prevents the auger from wobbling. Double-flight augers are usually used for excavating stronger geomaterials than are excavated with single-flight augers. Some augers may be true double flight augers, as on the right of Figure 4-16, and some may have a “dummy flight” to provide a double cutting surface but feed the cuttings into a single auger flight. The stinger for a single-flight auger is typically more substantial than for an auger with a double cutting surface because the single-flight auger must sustain a greater unbalanced moment during cutting. Double flight augers are generally preferred for large diameter holes (Figure 4-17) so that the cutting resistance on the base of the tool is more evenly balanced. Some contractors have found that double-flight augers without stingers can be used efficiently.

The flighting for augers must be carefully designed so that the material that is cut can move up the auger without undue resistance. Some contractors have found that augers with a slight cup shape are more effective at holding soils when drilling under slurry than standard non-cupped augers. The number and pitch of the flights can vary widely. The type of auger, single-flight or double-flight, cupping, and the number and pitch of flights will be selected after taking into account the nature of the soil to be excavated. The length of the auger affects the amount of material that may be excavated in one pass, and the maximum length may be limited by the torque and/or lifting capability of the drilling machine. Longer augers also tend to drill straighter holes, but are heavier to hoist.

The cutting face on most augers is such that a roughly flat base in the borehole results (that is, the cutting face is perpendicular to the axis of the tool). The teeth in Figures 4-15 and 4-16 are flat-nosed for excavating soil or decomposed rock, whereas the rounded teeth in Figure 4-17 are for ripping harder

material. The shape and pitch of flat-nosed teeth can be varied; modifying the pitch on auger teeth by a few degrees can make a significant difference in the rate at which soil or rock can be excavated, and the contractor may have to experiment with the pitch and type of teeth on a project before reaching optimum drilling conditions.



Figure 4-15 Single Flight Earth Augers



Figure 4-16 Double Flight Earth Augers



Figure 4-17 Large Diameter Auger with Double Cutting Edge

An important detail, particularly in soils or rock containing or derived from clay, is that softened soil or degraded rock is often smeared on the sides of otherwise dry boreholes by augers as the cuttings are being brought to the surface in the flights of the auger. This smeared material is most troublesome when some free water exists in the borehole, either through seeps from the formation being drilled or from water that is introduced by the contractor to make the cuttings sticky for facilitating lifting. Soil smear can significantly reduce the side resistance of drilled shafts, particularly in rock sockets. A simple way to remove such smear is to reposition the outermost teeth on the auger so that they face to the outside, instead of downward, and to insert the auger and rotate it to scrape the smeared material off the side of the borehole prior to final cleanout and concreting.

Care must be exercised in inserting and extracting augers from columns of drilling fluid, as the fluid is prone to development of positive (insertion) and negative (extraction) pressures that can destabilize the borehole. The addition of teeth on the side of the auger to excavate a hole larger than the size of the tool can be beneficial in allowing slurry to pass. The tool may also be equipped with one or more slurry bypass ports; the tool shown in the foreground of Figure 4-18 incorporates a slurry bypass sleeve around the kelly connection.

Cobbles or small boulders can sometimes be excavated by conventional augers. Modified single-helix augers (Figure 4-19), designed with a taper and sometimes with a calyx bucket mounted on the top of the auger, called "boulder rooters," can often be more successful at extracting small boulders than standard digging augers. The extraction of a large boulder or rock fragment can cause considerable difficulty, however. If a boulder is solidly embedded, it can be cored. When boulders are loosely embedded in soil, coring may be ineffective. The removal of such boulders may require that the boulders be broken by impact or even by hand. A boulder can sometimes be lifted from the excavation with a grab, or by cable after a rock bolt has been attached.



Figure 4-18 Auger with Slurry Bypass

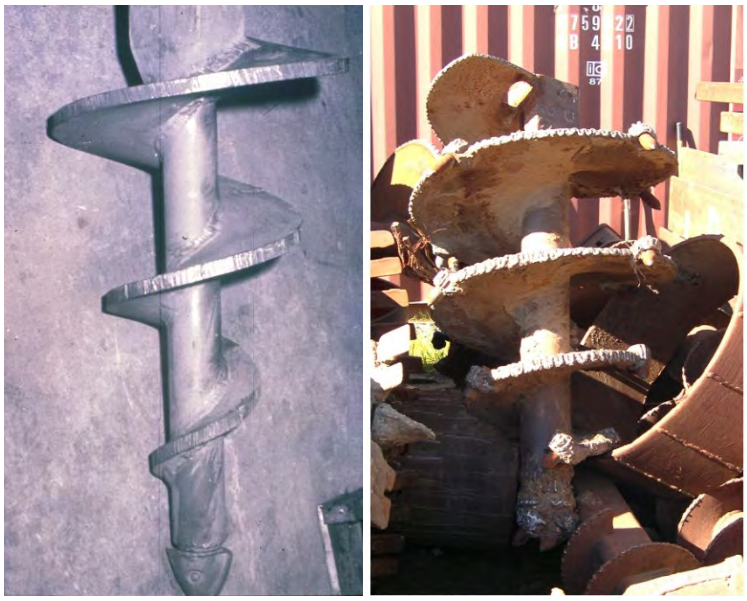


Figure 4-19 Boulder Rooters

4.3.1.1.2 Rock Augers

A flight auger specially designed for rock can be used to drill relatively soft rock (hard shale, sandstone, soft limestone, decomposed rock). Hard-surfaced, conical teeth, usually made of tungsten carbide, are used with the rock auger. Rock augers are often of the double-helix type. Three different rock augers are shown in Figure 4-20. As may be seen in the figure, the thickness of the metal used in making the flights is more substantial than that used in making augers for excavating soil. The geometry and pitch of the teeth are important details in the success of the excavation process, and the orientation of the teeth on a rock auger is usually designed to promote chipping of rock fragments. Rock augers can also be tapered, as shown in Figure 4-20.

Some contractors may choose to make pilot holes in rock with a tapered auger of a diameter smaller than (perhaps one-half of) that of the borehole. Then, the hole is excavated to its final, nominal diameter with a larger diameter, flat-bottom rock auger or with a core barrel. The stress relief afforded by pilot-hole drilling often makes the final excavation proceed much more easily than it would had the pilot hole not been made. It should be noted that tapered rock augers will not produce a flat-bottomed borehole, and an unlevel base in the borehole can be more difficult to clean and to produce a sound bearing surface.



Figure 4-20 Rock Augers

4.3.1.2 Drilling Buckets

Drilling buckets are used mainly in soil formations, as they are not effective in excavating rock. Soil is forced by the rotary digging action to enter the bucket through the two openings (slots) in the bottom; flaps inside the bucket prevent the soil from falling out through the slots. A typical drilling bucket is shown in Figure 4-21. After obtaining a load of soil, the tool is withdrawn from the hole, and the hinged bottom of the bucket is opened to empty the spoil. Drilling buckets are particularly efficient in granular soils, where an open-helix auger cannot bring the soils out.

They are also effective in excavating soils under drilling slurries, where soils tend to "slide off" of open helix augers. When used to excavate soil under slurry, the drilling bucket should have channels through which the slurry can freely pass without building up excess positive or negative pressures in the slurry column below the tool. It is often easier to provide such pressure relief on drilling buckets than on open-helix augers.

The cutting teeth on the buckets in Figure 4-21 are flat-nosed. These teeth effectively "gouge" the soil out of the formation. If layers of cemented soil or rock are known to exist within the soil matrix, conical, or "ripping," teeth might be substituted for one of the rows of flat-nosed teeth to facilitate drilling through alternating layers of soil and rock without changing drilling tools.



Figure 4-21 Typical Drilling Buckets

Drilling buckets are generally not appropriate for cleaning the bases of boreholes. Other buckets are designed to clean the base when there is water or drilling slurry in the hole (Figure 4-22). These are known as "muck buckets" or "clean-out buckets." Clean-out buckets have cutting blades, rather than teeth, to achieve more effective removal of cuttings and a more uniform bottom surface. The operation of the closure flaps on the clean-out bucket, or steel plates that serve the same purpose as flaps, are critical for proper operation of the clean-out bucket. If such flaps or plates do not close tightly and allow soil to fall out of the bucket, the base cleaning operation will not be successful. As with drilling buckets, clean-out buckets should be equipped with channels for pressure relief if they are used to clean boreholes under slurry.



Figure 4-22 Clean-out Buckets

4.3.1.3 Core Barrels

If augers are ineffective in excavating rock (for example, the rock is too hard), most contractors would next attempt to excavate the rock with a core barrel. Coring can be more effective in loading the individual cutting bits since the load is distributed from the crowd to the perimeter rather than to the entire face of the hole. Ideally, the tube cores into the rock until a discontinuity is reached and the core breaks off. The section of rock contained in the tube, or "core," is held in place by friction from the cuttings and is brought to the surface by simply lifting the core barrel. The core is then deposited on the surface by shaking or hammering the core barrel, or occasionally by using a chisel to split the core within the core barrel to allow it to drop out.

The simplest form of core barrel is a single, cylindrical steel tube with hard metal teeth at the bottom edge to cut into the rock, as illustrated in Figure 4-23. These simple core barrels have no direct means to remove rock chips from the cutting surface. The tools in these photos include a variety of cutting teeth positioned in a staggered pattern designed to avoid tracking in the same groove and to cut a hole slightly larger than the tool. The chisel teeth shown at bottom left would be used in soft rock, while the conical points shown at bottom right would be used in somewhat harder material. The “button” teeth shown at center right are used to cut harder rock where the conical points are prone to breaking off. Note also that the oscillator/rotator casing is a type of core barrel which commonly employs the button teeth, as shown in the top most photos.

If the rock is hard and a significant penetration into the rock is required, a double walled core barrel may be more effective. Double walled coring tools are more expensive and sophisticated, and can incorporate roller bits as well as teeth. Some examples are shown in Figure 4-24. The cuttings are removed by circulation of air if a dry hole is being excavated, or by circulation of water in a wet hole. The double wall provides a space through which the drilling fluid is pumped to the cutting surface. Double-walled core barrels are generally capable of extracting longer cores than single-walled core barrels, which constantly twist and fracture the rock without the provision of fluid to remove cuttings.



Figure 4-23 Single Wall Core Barrels



Figure 4-24 Double Wall Core Barrels

One of the problems with the use of the core barrel is to loosen and recover the core (Figure 4-25) after the core barrel has penetrated a few feet. Various techniques can be used for such a purpose. If the core breaks at a horizontal seam in the rock, drillers may be able to lift the core directly or by a rapid turning of the tool. Note the rock core contained within the barrel in the photo at bottom left of Figure 4-25. The photo at bottom right shows a hydraulically operated device for grabbing a core for extraction. When the core does not come up with the barrel, a chisel (wedge-shaped tool) can be lowered and driven into the annular space cut by the core barrel either to break the core off or to break it into smaller pieces for removal with another piece of equipment. Chisels and other percussion tools are described in a following section of this chapter. Blasting may also be employed to break up a core, where permitted. A hammergrab or clamshell can be used to lift loose or broken cores, if necessary.







Figure 4-25 Rock Cores

4.3.1.4 Full-Faced Rotary Tools

Full face rotary tools may be used for drilling rock, particularly at a large depth. Figure 4-26 shows some tools that are used for this purpose and which utilize roller bits that are attached across the entire cutting face of the tool. The roller bits grind the rock, which is transported to the surface by flushing drilling fluid with the reverse circulation technique described earlier. Disk shaped cutter heads or even teeth have been employed with full face tools in soft rock or cemented soils. Full face rotary tools have occasionally been used with direct circulation in small diameter holes (less than 30 inch) in hard rock by forcing compressed air down through the center of the drill string to blow cuttings out.



Figure 4-26 Full-Faced Rotary Cutters

4.3.1.5 Special Rotary Tools

Innovative equipment suppliers and contractors have developed a large number of special tools for unusual problems that are encountered. The tool on the left in Figure 4-27 cuts grooves in the walls of the borehole in order to facilitate development of the shearing strength of the soil or rock along the sides of the drilled shaft. The core barrel on the right in Figure 4-27 has been outfitted with steel wire on the outside of the barrel to scrape cuttings or loose rock (usually degradable shale) from the surface of rock sockets. Such devices are known regionally as "backscratchers." Other tools are used for assistance in excavation. For example, Figure 4-28 shows a drawing and photo of a tool (the "Glover Rock-Grab") that can core and subsequently grab rock to lift it to the surface. This tool is sometimes effective in excavating boulders or fragmented rock where augers or ordinary core barrels are unsuccessful. Numerous other special tools may be developed by equipment suppliers or contractors for specific projects.



Figure 4-27 Special Rotary Tools: Grooving Tool (left) and “Backscratcher” (right)



Figure 4-28 Boulder-Grabber Tool

4.3.2 Percussion and Other Tools

In contrast to rotary drilling, percussion drilling involves the breaking up of rock, boulders or cemented soil by impact. The broken material may be removed with a clamshell-type bucket or other means such as air circulation. The tools used with percussion methods range from the most simple and crude drop tools to sophisticated hammer drills.

4.3.2.1 Clamshell or Grab Bucket

Bucket excavation is initiated by the setting of a guide for the tools, a procedure that corresponds to the setting of a surface casing when rotary methods are being used. The guide may be circular or rectangular and is designed to conform to the excavating tool. The cross sections of such excavations can have a variety of shapes and can be quite large. With the oscillator or rotator systems (Section 4.2.3.5), the circular segmental casing serves as the guide. With the types of clamshell tools used to construct diaphragm walls or barrettes (Section 4.2.4), a guide-wall is often constructed at the ground surface.

Two types of lifting machines may be used to handle the digging tools that are needed for non-rotary excavation. The simplest procedure is to raise and lower the tools with a cable such as provided by a crane (the term "cable tool" is often used to describe tools used in this manner). The jaws of the digging bucket can be opened and closed by a mechanical arrangement that is actuated by a second cable or by a hydraulic system. The other type of lifting machine uses a solid rod for moving the excavating tools up and down. The rod, which may be called a kelly, is substantial enough to allow the easy positioning of the tool. The kelly in this case does not rotate but merely moves up and down in appropriate guides. As with the cable tool, a mechanism must be provided for opening and closing the jaws of the bucket.

Clamshell or grab buckets are often used in situations where rotary tools are unproductive or impractical. For example, a digging bucket can be used to excavate broken rock, cobbles and soils that are loose and that can be readily picked up by the bucket. If hard, massive rock or boulders are encountered, a tool such as a rock breaker may be used. The broken rock is then lifted using a clamshell or a grab bucket. A typical clamshell, with a circular section for use in drilled shafts, is shown on the left in Figure 4-29. Clamshells and grab buckets are available in various diameters up to about 6 ft. Clamshells or grab buckets can also be used to make excavations with noncircular cross sections, as shown at right in Figure 4-29. The transverse dimension of the tool must conform to the shape of the guides that are used.





Figure 4-29 Clam-shell Buckets

4.3.2.2 Hammergrabs

Hammergrabs are percussion tools that both break and lift rock. Examples of hammergrabs in use are shown in Figure 4-30. Hammergrabs are made heavy by the use of dead weight. The jaws at the bottom of the tool are closed when the tool is dropped, and the wedge formed by the closed jaws breaks the rock. The jaws have strong, hardened teeth and can open to the full size of the tool to pick up the broken rock. Hammergrabs are heavy and relatively expensive devices; however, they have the advantage over rock breakers and clamshells in that the tool does not need to be changed to lift out the broken rock, which speeds the excavation process. Hammergrabs can also be used to construct noncircular barrettes by changing the length of the long side.



Figure 4-30 Hammergrabs

4.3.2.3 Rock Breakers and Drop Chisels

These types of tools (Figure 4-31) are generally composed of a heavy object that is lifted and dropped to break up boulders, cores, and strong soils in order to break up the material and permit it to be lifted by a clamshell or a grab bucket. These tools may even be used to break rock at the bottom of the hole in order to advance the hole more easily. Several types of tools are made to be dropped by a crane. Chisels have a single point designed to help break off a core or to break off a boulder or ledge on the side of the hole. Some examples of rock breakers shown in Figure 4-31 are referred to as a "churn drill" or a "star drill." The bottom of these tools has a wedge shape so that high stresses will occur in the rock that is being impacted by the tool.

After the rock is broken, the broken pieces may be removed with a clam or hammergrab, or sometimes with a rotary auger.

4.3.2.4 Downhole Impact Hammers

To excavate hard rock across the full face of the shaft, a large diameter downhole hammer can be used in a drilling operation to make an excavation up to about 7 ft in diameter through very hard rock such as granite. Examples of downhole hammers are shown in Figure 4-32. The tool at left is a cluster of air-operated hammers, sometimes referred to as a “cluster drill.” Downhole hammers are typically employed for rock which has proven extremely difficult to remove by core barrels or other means. The debris is typically raised by the use of air (i.e., debris is blown out of the borehole) if the hole is dry. The excavation of rock in such a manner is obviously extremely expensive and rock sockets in such hard material are best avoided, especially in urban environments where rock dust can create a hazard.



Figure 4-31 Drop Chisels and Rock Breakers



Figure 4-32 Downhole Impact Hammers

4.3.2.5 Blasting

Blasting is usually not permitted for excavation of drilled shafts because of the safety hazard and because of the potential for fracturing of the surrounding bearing formation. Fractures in the rock around the shaft could be detrimental to the performance of the foundation.

Explosives may be considered on rare occasions to aid in hand excavation of rock near the surface. For instance, explosives might be used to break through a boulder or obstruction within the hole. Explosives might also be employed through small predrilled holes to help level a steeply sloping surface and allow a casing to be more easily seated, or through hard pinnacle limestone above the zone relied upon for capacity. Primer cord has reportedly been used successfully to break cores away in the shaft by wrapping the cord around the base of the core at the bottom of the kerf. This small shock is not thought to affect the surrounding rock. Highly expansive cements have on occasion been used as alternates to explosives by placing cement paste in small holes drilled using air tracks into rock to split the rock and permit it to be excavated easily.

Explosives must be handled by experts and should be used only with the permission of the regulating authorities.

4.4 OTHER TECHNIQUES

4.4.1 Tools for Cleaning the Base of the Drilled Shaft Excavation

Other than the cleanout buckets described in Section 4.3.1.2, other non-rotary tools may be very useful for removing cuttings and debris from the base of the shaft. Most common is some type of pump to lift cuttings for removal. Figure 4-33 illustrates two types of pumps used to lift cuttings. The one on the left is an air-lift pump which operates by pumping air down the supply line alongside the air-lift pipe. As the air enters the pipe a few feet above the bottom, the rising column of air lifts the fluid within the pipe. The buoyant lifting of this column causes suction at the bottom of the pipe which will lift sand or loose material. The photo on the right of Figure 4-33 is of a hydraulic pump, which operates via the two hydraulic lines to rotate the impeller that pumps fluid upward from the base of the pump. Hydraulic pumps are more controllable than the airlift system in that the volume and velocity of pumping can be regulated more easily. Airlifts tend to remove larger particles than pumps.



Figure 4-33 Airlift and Hydraulic Pumps for Shaft Base Cleanout

Note that while pumping systems as shown in Figure 4-33 are probably the most effective means of removing loose cuttings or debris from the base of a wet hole, the aggressive use of these tools in cohesionless sands can advance the shaft excavation. The effectiveness of shaft base cleanout tools and techniques in wet holes can best be evaluated using a downhole camera, as described in Section 15.2.4.

4.4.2 Methods for Stabilizing Soils or Formation

On rare occasions, permeation or compaction grouting may be employed to stabilize a particularly unstable stratum or around a very large or deep hole. Baker et a1. (1982) report that grouting in advance of excavation can sometimes be used to reduce water inflow effectively and even to permit construction of under-reams in granular soil. Examples were given where the technique was used successfully in Chicago.

The principles and use of grouting to enhance base resistance in granular soils was described in Chapter 3. Skin grouting through sleeve-port tubes along the side of the shaft has been used on rare occasions internationally, but is rarely employed in U.S. practice.

4.5 SUMMARY

This chapter described a variety of drilling machines and tools used for construction of drilled shafts. In recent years, the variety of machines and tools has increased as the industry matures and equipment becomes more specialized. The specific choice of tools and equipment is generally the responsibility of the contractor, and different constructors may approach the project differently depending upon their personal experience and resources. Engineers charged with design, specification, and inspection of drilled shafts must be knowledgeable of drilling equipment in order to provide appropriate specifications, and a constructable and cost efficient design. Sufficient subsurface information must be provided so that bidders can make an informed decision about equipment to use on the job. The tools and equipment that are planned for use by the contractor should be described in the contractor's drilled shaft installation plan and the equipment actually used documented in the construction records.

Temporary or permanent casings are additional tools that may be used to complete the drilled shaft, as described in detail in Chapter 5.

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CHAPTER 5
TECHNIQUES FOR MAINTAINING STABLE DRILLED SHAFT EXCAVATIONS

The stability of the drilled shaft excavation must be maintained during the entire construction process from the start of drilling to the completion of concrete placement and removal of temporary casing. A stable excavation is critically important for several reasons. First and most importantly, the safety on the jobsite relies on a stable hole because a collapse could undermine the supporting platform for workers and equipment near the hole. Ground movements related to instability of the excavation can potentially impact nearby structures as well as the integrity of the bearing formation into which the drilled shaft is constructed. Sloughing or collapse of an unstable hole during placement of reinforcement and concrete can jeopardize the structural integrity of the completed foundation, as well as the geotechnical available resistance.

Some ground conditions are very favorable to easy drilled shaft construction because of their inherent natural stability, but, more typically, the use of steel casings, drilling fluids, or some combination of both, are required to maintain stability of the shaft excavation.

This chapter provides an overview of the various techniques that may be employed to maintain the stability of the drilled shaft excavation.

5.1 NATURALLY STABLE CONDITIONS

When ground conditions are inherently stable, as illustrated in Figure 5-1, excavation for drilled shaft construction can often proceed without the need for additional support. These conditions are typically limited to:

- Dry soil (little or no seepage into the hole)
- Strong cohesive or cemented soils
- Intact rock

Of course, the relative strength of the soil needed to provide construction stability depends on factors such as the diameter and depth of the hole, the loads on the ground surface nearby, potential vibrations during construction, and the length of time the hole must remain open. Local experience often provides a good guide, but does not eliminate the need for site-specific geotechnical information. Many developed areas of the southwestern U.S. (e.g. parts of Texas, Colorado, and Nevada) have conditions which are conducive to open-hole excavation, and in such conditions drilled shafts can be constructed very economically.

In some cases the soil may have inherent stability for a portion of the construction process, but additional measures may be needed at the start or completion of the drilled shaft. An example might include a drilled shaft which is to be constructed through cohesive soil with an extended embedment into granular soil or rock. In this circumstance, the portion of the excavation extending through the cohesive overburden may be performed as an open, dry hole, after which additional measures are employed for the remainder of the excavation. For example, it may be possible to excavate using an open hole through overburden soils to the top of rock or strong layer before seating a steel casing into the stronger material and completing the installation within a dry, stable excavation below the casing (see Figure 5-2). If the

strong layer has very low permeability (such as shale, chalk, or massive intact rock), then it may be possible to excavate a dry hole even below the groundwater surface.



Figure 5-1 Dry Cohesive or Cemented Soils May Provide Naturally Stable Ground Conditions



Figure 5-2 Inserting Casing into an Open Hole Prior to Rock Excavation at a Site in Georgia

Rock, even formations with moderate groundwater conductivity, and cemented materials may provide inherent stability even below the groundwater. Examples include porous limestone or permeable sandstone. In general, however, it is desirable to add water to the hole during the excavation of such materials rather than allow flow from the ground into the hole, which can bring fines or cause sloughing of weak layers, and degrade the structural integrity of the drilled shaft concrete.

Although cohesive soils with low permeability may appear to provide a dry excavation, it is important to anticipate the requirements of subsequent strata. For example, advancing a dry excavation through a clay soil into a water-bearing sand stratum could result in "flowing" sands or bottom heave if the groundwater level in the sand is higher than the base of the excavation.

5.2 TEMPORARY CASING

Temporary casing is used to stabilize the drilled shaft excavation and then removed after or during placement of fluid concrete. Contractors like to emphasize the fact that the casing that is used temporarily in the drilling operation is essentially a tool, so it is sometimes termed "temporary tool casing." The temporary casing remains in place until the fluid concrete has been placed to a level sufficient to withstand surrounding ground and groundwater pressures, and then is removed. Additional concrete may be placed as the casing is being pulled to maintain the pressure balance. The fluid pressure of the concrete is relied upon to provide borehole stability. The use of temporary casing has been described briefly in Chapter 3.

When approved by the engineer, temporary casing used solely for support of the drilled shaft excavation may be left in place. In such cases, the engineer must assess the influence of the casing on the axial and lateral resistance of the completed drilled shaft. In some soils, such as soft cohesive soils that may be prone to squeezing or bulging if the casing is removed, the engineer may require the temporary casing to remain permanently in place through potentially unstable soil layers.

Temporary casing must be cleaned thoroughly after each use to have low shearing resistance to the movement of fluid concrete. Casing with a rough interior surface may drag on the column of concrete as the casing is lifted, cause necking or voids in the shaft, displace the reinforcement, or even cause the casing to become stuck. The casing should be free of soil, lubricants and other deleterious material.

5.2.1 Types and Dimensions

Most drilling contractors will maintain a large supply of temporary casing of various diameters and lengths in their construction yards. A typical view of stored temporary casing is shown in Figure 5-3. Casing from the stockpile may be welded or cut to match the requirements of a particular project.

Temporary casing must sometimes be seated into an impervious formation such as rock if the excavation is to be advanced below the casing in the dry. In such a circumstance, it will normally be necessary to use the casing as a tool, with twisting or driving forces applied through the casing. The end of the casing may be equipped with cutting teeth or additional thickness in order to facilitate installation and avoid distorting the casing.



Figure 5-3 A Typical View of Stored Temporary Casing

ADSC: The International Association of Foundation Drilling, has adopted the outside diameter of casing as a standard and uses traditional units [e.g. 36-in. O.D.] because used pipe in O.D. sizes is available at much lower cost than specially rolled pipe with specified I.D. (ADSC, 1995). Specially ordered pipe of a specific size can be ordered, but at higher cost and with the added requirement of lead time for fabrication. Ordinarily, O.D. sizes are available in 6-inch increments from 18 to 120 inches. Larger sizes, as shown in Figure 5-4, typically require special order and fabrication.

It is also noted that the use of manufactured segmental casing typically comes in metric sizes because of the worldwide distribution of this equipment. It is therefore advisable that the design include some flexibility on sizes where the use of temporary casing is anticipated in order to facilitate the cost-effective use of segmental casing. For example, 1.2 m (47.25 in.) diameter for 48-in. size, or 3 m (118 in.) diameter for a 10-ft (120-in.) size can normally be accommodated with small adjustments in the acceptable sizes, otherwise the contractor would be forced to upsize the diameter at considerable additional cost to employ segmental casing.

If the temporary casing size is not specified, most contractors will usually employ a casing that has an O.D. that is 6 inches larger than the specified drilled shaft diameter below the casing to allow for the passage of a drilling tool of proper diameter during final excavation of the shaft. A drilling tool with a diameter equal to the specified shaft diameter below the casing will usually be used. If boulders are anticipated, or if the contractor otherwise decides to use telescoping casing, the first casing that is set may have an O.D. that is more than 6 inches larger than the specified shaft diameter.

The contractor is usually responsible for selecting a casing with sufficient strength to resist the pressures imposed by the soil or rock and internal and external fluids. Most steel casing has a wall thickness of at least 0.325 inches, and casings larger than 48 inch O.D. tend to have greater wall thicknesses. Installation with vibratory or impact hammers may require greater wall thickness than would be used for casing installed in an oversized hole. Most contractors rely on experience in the selection of casing wall thickness. However, if workers are required to enter an excavation, the temporary or permanent casing should be designed to have an appropriate factor of safety against collapse.



Figure 5-4 Exceptionally Large Temporary Casings

The computation of the allowable lateral pressure that can be sustained by a given casing is a complex problem, and methods for such computations are beyond the scope of this publication. The problem is generally one of assuring that buckling of the casing does not occur due to the external soil and water pressures. Factors to be considered are: diameter, wall thickness, out-of roundness, corrosion, minor defects, combined stresses, microseismic events, instability of soil on slopes and other sources of nonuniform lateral pressure, and lateral pressure that increases with depth.

Semi-rigid liners can be used for liners or surface casing that may be left in place. They can consist of corrugated sheet metal, plain sheet metal, or pressed fiber. Plastic tubes or tubes of other material can also be used. These liners are most often used for surface casing where it is desirable to restrain unstable surface soil that could collapse into the fluid concrete during placement, or to facilitate construction of the drilled shaft to column connection. For example, corrugated sheet metal is often used for this purpose when the concrete cutoff elevation is below working grade. Occasionally, rigid liners, such as sections of precast concrete pipe, are also used effectively for this purpose.

Rotators and/or oscillators with segmental casing (Figure 5-5) are increasingly being used to advance large diameter, deep drilled shafts. The casing penetration is advanced ahead of the excavation, thus providing support for the excavation and eliminating the need for slurry for side wall stability. However, slurry or water may still be necessary to prevent base heave. Soil can be removed within the casing with clam, hammer-grab, or rotary tools. The casing is typically high strength steel, often double-wall, with flush fitting joints between segments. Details of the connection between casing segments allow for the transmission of torque, compression, and tension between casing sections. This allows large torque (in either direction), compression, and lifting forces applied by equipment at the surface to be transmitted from the top section of casing to the bottom section of casing.

Although the double walled casing shown in Figure 5-5 is most often used, it is possible to weld the casing joints to standard pipe as illustrated in Figure 5-6. In this case the casing joint will protrude into the interior of the casing.



Figure 5-5 Segmental Casing Installation with Oscillator System



Figure 5-6 Installation of Casing Joint on Standard Pipe

5.2.2 Installation and Extraction of Temporary Casing

As described in Section 5.1, temporary casing is sometimes placed into an oversized drilled hole and then seated into the underlying formation to provide a stable environment, but temporary casing can also be advanced ahead of or in combination with the advancement of the excavation. Each of these methods

may have implications for design because of the effect on the side resistance in the zone through which the temporary casing is installed. Methods for installation and extraction of temporary casing are described below.

5.2.2.1 Casing Seated Through Drilled Hole

Temporary casing can be placed through a pre-drilled hole to seat the casing into an underlying formation of more stable material. The pre-formed hole may be constructed using the wet method with drilling slurry, or may sometimes be advanced without a drilling fluid if the soil will stand for a short period and the seepage into the hole is relatively small. The latter is often the case where the shaft excavation can be drilled relatively quickly through a residual soil to rock; then the more time-consuming rock excavation is facilitated by having a temporary casing to prevent cave-ins of the overlying soil. If the shallow strata are water-bearing sands, it may be necessary to drill the starter hole with slurry to prevent caving. In some instances, contractors may use polymer slurry just to help "lubricate" the casing and make it easier to remove.

The excavation below the casing may be advanced as a dry hole if the casing is seated with a watertight seal into a relatively impermeable underlying formation of clay, chalk or rock. In order to seat the casing, a "twister bar" attachment to the kelly bar may be used to allow the drill rig to apply torque and crowd to the casing and advance it into the underlying soil or rock. Figure 5-7 illustrates casings with J-slots cut into the top to allow a casing twister to be used. In order to help the casing to cut into the underlying formation, the end of the casing is usually equipped with cutting teeth as shown in Figure 5-8. Various types of cutting teeth may be used, depending upon the type of material into which the casing is advanced.



Figure 5-7 J Slots in Top of Casing for Use with Casing Twister



Figure 5-8 Teeth for Use in Sealing Casing into Rock (Photograph at top left courtesy of Herzog Foundation Drilling, Inc.)

A good seal of casing into underlying rock can be very difficult if the rock surface is steeply sloping or highly irregular, or if the rock contains seams or joints that allow water inflow below the casing. It is therefore often necessary that the casing be advanced some distance into the rock; accordingly, in such conditions, the foundation design should include some flexibility with regard to the casing tip elevation. An irregular hard surface presents risks for the casing to deflect off alignment, break cutting teeth, and possibly bend the casing due to concentrated stresses if excessive downforce is applied.

Drilled shaft excavations can be made using more than one piece of casing with the "telescoping casing" process (Figure 5-9). This process has the economic advantage that smaller cranes and ancillary equipment can be used to install and remove telescoping casing than would be required with a single piece of casing. A borehole with a diameter considerably larger than that specified is made at the surface, and a section of casing is inserted. A second borehole is excavated below that section of casing, which is then supported with another section of casing of smaller diameter. This process may proceed through multiple, progressively smaller casings, with the I.D. (O.D- if excavating does not proceed below casing) of the lowest casing being equal to or greater than the specified diameter of the drilled shaft. The O.D. of a lower section of such "telescoping casing" is typically at least 6 inches smaller than the O.D. of the section above it, although larger differential diameters may be used when necessary. This procedure is most often used for drilled shafts that are bearing on or socketed into rock and where no skin friction is considered in the soils or rock that is cased. Care must be taken by the contractor that the process of removing the smaller section(s) of casing does not disturb the larger section(s) of casing still in place, or deposit water, slurry or debris behind casings still in place, thereby contaminating the fluid concrete.

Telescoping casing may also be used to case through boulder fields where some boulders are removed as the casing is screwed ahead to refusal. The smaller inner casing is advanced through the first casing which retains the zone where the larger boulders were removed. The placement of concrete within a hole stabilized using telescoping casing is described in Chapter 7.



Figure 5-9 Use of Telescoping Casing to Complete a Dry Excavation at a Site Near Dallas, Texas

5.2.2.2 Casing Advanced Ahead of Excavation

The contractor may choose to advance the casing ahead of the excavation in cases where the hole will not stand open for short periods or where slurry drilling techniques are considered less attractive from a cost or performance standpoint. There are two primary methods used to advance casing ahead of the excavation. The contractor may drive the casing in advance using a vibratory hammer, or by twisting using the drill rig or using oscillator/rotator equipment.

In either case it is important to note that the friction of the soil acting against the side wall of the casing must be reduced to a sufficient degree that the casing can be advanced. The vibratory hammer accomplishes this reduction by temporarily reducing the strength of the soil along the sidewall. In order to twist the casing into the soil, a reduction in friction is achieved by a small overcut achieved with the casing shoe or cutting teeth. These techniques allow the soil to "arch" around the circular drilled shaft excavation; the horizontal stresses acting in the radial direction against the side of the casing are thereby transferred into hoop stresses arching around the hole, as illustrated in Figure 5-10.

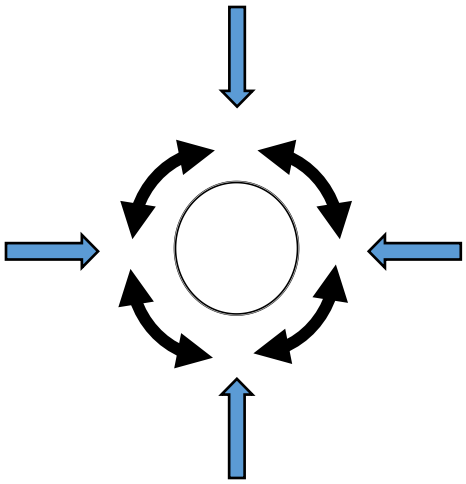


Figure 5-10 Reduced horizontal stress due to soil arching, which develops as soil moves inward

5.2.2.2.1 Vibro-Driven Casing

When casing is driven, a vibratory hammer is almost always used for temporary casing; an impact hammer may be used to install permanent casing, but temporary casing will require a vibratory hammer for extraction since casing installed with an impact hammer may be impossible to remove due to increased friction resistance. In principal, jetting could be utilized as an aid to installation, but jetting around the casing would not be advised during extraction due to the potential for jet water to adversely affect the fluid concrete.

In planning the construction of drilled shafts in congested areas, it should be noted that the use of vibratory installation of casing can cause significant vibrations that can affect nearby structures, or cause settlement in loose sands (which can affect nearby structures and utilities). The attenuation of vibrations with distance away from the source is affected by the size of the hammer and casing, the operating frequency of the hammer, the soil and rock properties, the localized stratigraphy, groundwater, and other factors that are likely site-specific. In most cases, vibrations from casing installation are extremely small at distances of 50 to 70 ft from the source, but may extend to greater distances when penetrating hard or cemented layers. In cases where sensitive structures may be present nearby, a program of vibration monitoring should be included in the installation plan. Vibration monitoring can help avoid potential damage and can also provide documentation as protection against lawsuits or claims of damage caused by vibratory installation of casing. Monitoring during construction of the technique and test shaft installations can provide valuable measurements of vibrations at various radial distances from the source before moving the work into more congested production locations. A useful reference on this subject is "Construction Vibrations" by Dowding (2000).

Installation of the casing using a vibratory hammer is most effective in sandy soil deposits, and to penetrate through sandy soils into a clay or marl stratum below. The hammer clamps to the top of the casing (Figure 5-11), which is often reinforced at the end with an extra thickness to aid in resisting the transmitted forces. The vibration of the casing often causes temporary liquefaction of a thin zone of soil immediately adjacent to the casing wall so that penetration is achieved only with the weight of the casing plus the hammer. This technique is particularly effective in sandy soils with shallow groundwater. Penetration of an underlying hard layer such as cemented sand or rock may be difficult or impossible with a vibro-driven casing. Attempts to twist the casing with the drill rig to seat into rock are likely to be ineffective because of the side resistance of the soil against the casing after removal of the vibration.

In general, a vibratory hammer is used to place the entire length of temporary casing into the soil before excavation of soil inside the casing. However, to facilitate penetration through particularly dense soils, the casing can be installed by an alternating sequence of driving the casing and drilling to remove the soil plug within the casing. In this case, it would typically be necessary to install the casing in sections, with the sections joined by welding.

Another technique which might be employed is referred to as "relief drilling" whereby an auger is advanced through a hard layer below the casing to break up the material so that the casing can subsequently be advanced through it. If the soil below the casing is not considered sufficiently stable to stand open prior to advancing the casing deeper, the rotation of the auger can be reversed on extraction to leave the soil in place below the casing to avoid having an uncased hole below the casing.

Removal of the casing with the vibratory hammer must be accomplished while the concrete is still fluid. During extraction, the hammer is attached and powered, and then typically used to drive the casing downward a few inches using the weight of the casing and hammer to break the casing free of the soil. Once the casing is moved, the crane pulls the casing upward to remove it and leave the fluid concrete filled hole behind. The photo in Figure 5-11 shows the start of removal of a casing after completion of concrete placement.



Figure 5-11 Extraction of Temporary Casing Using a Vibratory Hammer

5.2.2.2.2 Twisting/Rotating Method

Installation of temporary casing ahead of the excavation may be accomplished with a drill using special casing and tools, with a cutting shoe designed to minimize the soil friction on the casing. Because of the torque required to twist casing and overcome the soil resistance, the use of conventional drill rigs for this purpose is limited to relatively small diameter casing generally less than 4 ft diameter. Some of the hydraulic fixed mast rigs with a rotary drive system that can move vertically on the leads are well suited for this application, as illustrated in Figure 5-12 where the rig was used to install 39 inch diameter tangent piles for a retaining wall.



Figure 5-12 Segmental Casing Installed with a Hydraulic Drill Rig

The torque required to install larger diameter casings by twisting or rotating typically demands a specialized oscillator or rotator machine; a general description of the machines and procedures used for the oscillator/rotator method of construction is provided in Chapter 4. The oscillator or rotator clamps onto the casing with powerful hydraulic jaws and uses hydraulic pistons to twist the casing and push it downward, reacting against a large drilling machine or temporary frame. Figure 5-13 illustrates an oscillator attached to the drill rig, which installed temporary segmental casing to the top of rock prior to drilling the rock socket. The soil may also be excavated using a grab tool as illustrated in Figure 5-5.



Figure 5-13 Segmental Casing Installed with an Oscillator Attachment to a Hydraulic Drill Rig

Where casing is twisted or rotated to advance into soil or rock ahead of the excavation, the casing is provided with cutting teeth extending slightly beyond the outside dimension of the casing. The bottom section of casing is fitted with a cutting shoe to promote penetration (Figure 5-14) by cutting a slightly oversized hole to relieve the stress against the sides of the casing. The soil on the interior of the casing is excavated concurrent with casing installation to remove the resistance of this portion of the soil.



Figure 5-14 Cutting Shoe for Segmental Casing

During installation of the casing, it is essential that a plug of soil remain inside the casing so that the bottom of the excavation does not become unstable during installation. The thickness of soil within the casing may vary depending upon the strength and stability of the material at the base of the casing. In water-bearing soils, the head of water inside of the casing must also be maintained so that bottom heave does not occur. It is possible to use polymer or mineral based drilling fluids inside the casing to maintain stability, but the need for these fluids is usually avoided by maintaining a soil plug and, when needed, a head of water. It is necessary to maintain stability during installation because heave of soil into the casing would cause loosening of the ground around the excavation with adverse effects on side resistance and possible subsidence around the shaft.

At completion of the excavation, the soil plug may be removed to the base of the casing (or below) if the casing is extended into rock or a stable formation, or if a head of drilling fluid is used to maintain stability. If the hole terminates in water-bearing soil with only a water head for stability, it may be necessary that the casing extend below the base of the final excavation to avoid instability at the base. However, this procedure may result in an annular zone of loosened soil at the base of the drilled shaft excavation.

The thicker casing (typically about 2 to 2.5 inches) used with this method of construction is a consideration in selection of the cover and the spacers on the reinforcement cage. If a single wall pipe is used with the casing joints as shown in Figure 5-6, the joints will protrude inside the casing because the joint is typically thicker than the pipe. In such a case, the reinforcing cage will need to be fabricated and placed carefully so that nothing hangs on the casing joints during installation of the cage and/or extraction of the casing during concrete placement.

To avoid potential torsional deformation of reinforcement, the casing is typically oscillated back and forth during extraction, even if a continuous rotation was used during installation. The casing is typically extracted simultaneously as concrete is placed into the excavation, and concrete head above the tip of the casing must be maintained so that a positive concrete pressure is provided against the hole. If exterior groundwater pressure is present, the head of concrete and water inside the casing must exceed the exterior water pressure to prevent inflow of water and contamination of the concrete. It is also essential that the concrete remain fluid so that the oscillation of the casing does not transfer twisting forces into the reinforcing cage and cause distortion of the cage.

5.2.3 Possible Effects of Temporary Casing on Axial and Lateral Resistance

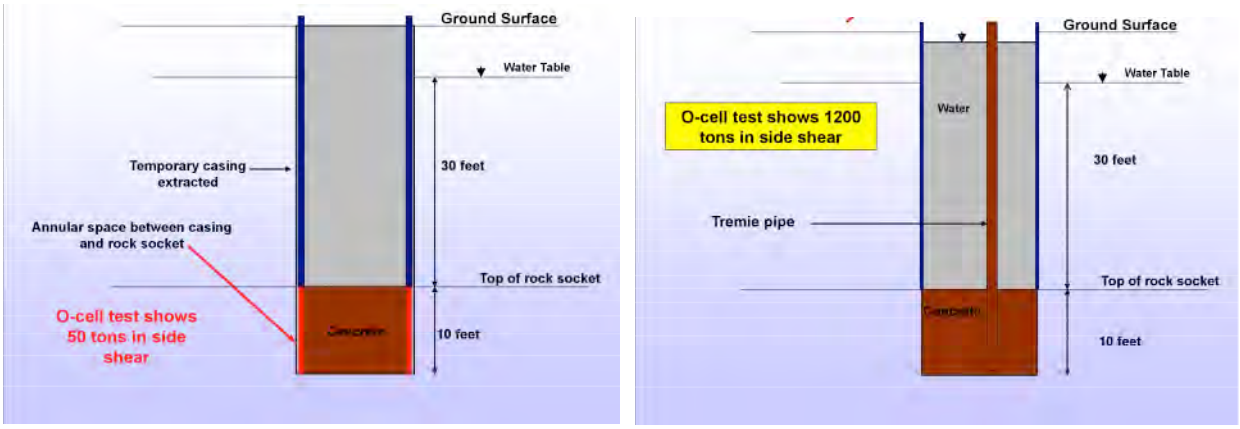
If temporary casing is to be used in construction, it is appropriate that the designer consider the possible effects of casing on axial and lateral resistance.

In general, no deleterious effect on the lateral resistance of a drilled shaft should occur due to the removal of temporary casing. However, an exception may be where the casing is installed in an oversized, pre-drilled hole that collapses into the void outside the casing prior to concrete placement and casing extraction, resulting in a zone of loosened soil around the casing. Temporary casing which cannot be removed and is left in place in an oversized hole may also reduce the lateral stiffness of the shaft due to the void; in this case, it is recommended that the void be filled with grout or flowable cementitious material to ensure transfer of lateral soil resistance around the shaft even if there is no reliance on the cased zone for axial resistance.

However, the axial resistance can be significantly affected by the method of casing installation, as discussed below.

If the axial resistance of the drilled shaft is derived entirely from the soil or rock below the temporary casing, there is little concern regarding any adverse effects of the casing on load transfer in side resistance. Designers should consider the relative magnitude of the contribution to axial resistance derived from the temporary casing zone; if this contribution is relatively small compared to the drilled shaft below this level, then it is appropriate and cost-effective to ignore the axial resistance of this portion of the shaft so that the constructor can be permitted to use the most cost-effective strategy to install the drilled shaft. If the side resistance of the temporary cased zone is significant, then there are important considerations as outlined below.

Casing installed into a predrilled hole may affect side resistance within the cased portion of the shaft if contaminants or debris or loosened soil are trapped behind the casing and are left between the concrete and native soil or rock. Contaminants can become trapped if thick, heavy slurry is used and left in the annular space behind the casing, or as a result of the casing being extracted too quickly, before the slurry can be effectively displaced by the flow of concrete. In addition, debris can fall into this annular space. Where temporary casing is installed into rock via a predrilled hole, it is likely that debris will collect in this space and a good concrete to rock bond will not be developed. An example of this problem is reported by Osterberg and Hayes (1999), and illustrated in Figure 5-15. A shaft was constructed using a casing extending the full length into a 10-ft deep rock socket to provide a dry excavation so that the base of the shaft in rock could be inspected. A bi-directional load test (described in Chapter 16) performed on the completed drilled shaft measured only 50 tons of side resistance in the rock socket, presumably because of trapped debris between the concrete and rock along the sidewall of the socket. At another drilled shaft, constructed by terminating the casing above the rock and constructing the rock socket "in the wet" under water, the load test measured 1200 tons of side resistance in the rock socket. This extreme example illustrates the importance of a simple detail in constructing drilled shafts into rock with casing.



a) Dry hole with temporary casing through the rock section b) wet hole with casing section at top of rock

Figure 5-15 Adverse Effect of Casing Extended into Rock Socket (Osterberg and Hayes, 1999)

Temporary casing installed ahead of the shaft excavation using a vibratory hammer should generally have no adverse effect on soil resistance in sands and can even have a beneficial effect by densifying the sand around the drilled shaft. However, a casing installed into and then extracted from a cohesive soil with a vibratory hammer is likely to result in a relatively smooth surface compared to a rough drilled hole. Camp et al. (2002) noted the relatively lower side resistance of the upper portion of a marl formation when temporary casing was used compared to an uncased shaft drilled with slurry, a difference which was attributed to the smoother shaft surface.

Segmental casing advanced ahead of the drilled shaft excavation using the oscillator/rotator system is generally considered to have no significant effect on side resistance so long as a stable excavation is maintained. The use of cutting teeth on the bottom of the casing and the oscillation of the casing during withdrawal tends to leave a rough surface texture on the drilled shaft, as reported by Brown (2012) and illustrated in Figure 5-16. Comparative tests by Brown (2002) and others reported by Katzenbach et al. (2008) suggest that this rough texture and other factors contribute to reasonably good unit side shear for drilled shafts constructed with this method compared to slurry methods, and possibly improved performance relative to shafts constructed using bentonite slurry. Since construction of very large or deep drilled shafts with bentonite slurry can be difficult to accomplish within the short time frame needed to avoid bentonite contamination at the interface, full length segmental casing can be a more favorable option for achieving the design side resistance. However, failure to maintain stability at the base of the excavation can result in loosening of the soil around the shaft excavation and a reduction of side resistance.

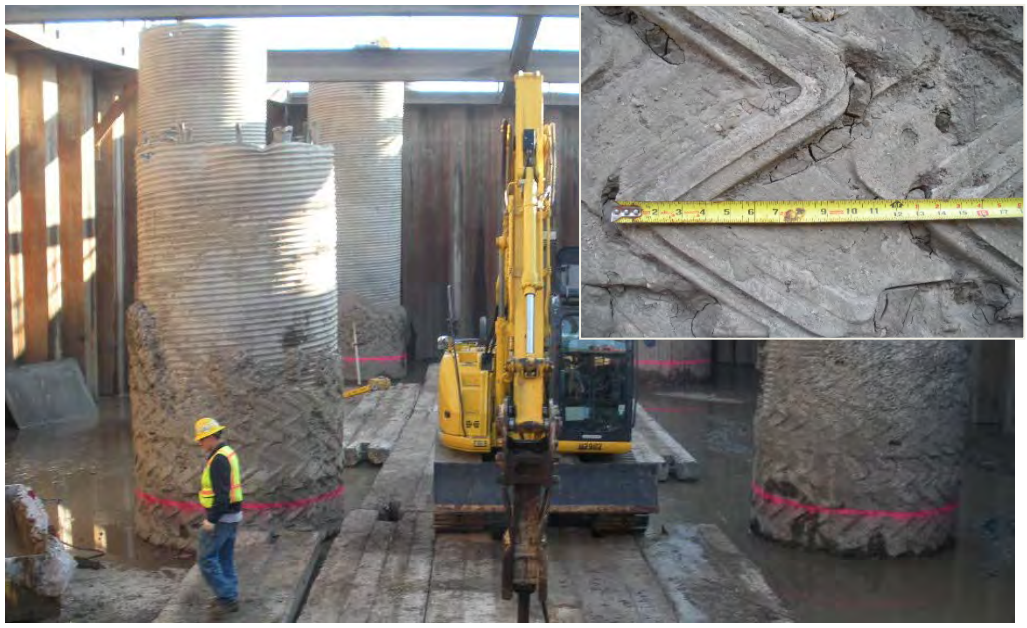


Figure 5-16 Exposed Surface of Drilled Shafts Constructed Using Oscillated and Rotated Casing

For cases where a temporary segmental casing extends into soil below the final bottom of the drilled shaft (as described in Section 5.2.2.2.2), an annular zone of loosened soil may form below the base of the drilled shaft. This disturbed annular zone may result in slightly reduced base resistance unless corrected by base grouting beneath the completed drilled shaft.

If the constructor is unable to extract the temporary casing, the responsible engineer needs to apply judgment to evaluate the effect of the casing on the axial resistance of the drilled shaft. Expedient load-testing methods, such as those described in Chapter 13, may be helpful in evaluating side resistance around casings that are unintentionally left permanently in place. Although it is not possible to make general statements that apply to all cases, many studies have been conducted that show that the load transfer from the casing to the supporting soil can be significantly less than if concrete had been in contact with the soil (Li et al, 2017, Lo and Li, 2003; Owens and Reese, 1982).

5.2.4 Removing Casing after Concrete Sets

Drilled shafts installed through a body of water typically use a permanent casing that serves as a form until the concrete sets, and then is left permanently in place. It is preferable to leave the casing in place rather than to attempt to remove it for several reasons. Firstly, the casing provides additional corrosion protection for the reinforcement, since chlorides must penetrate not only the concrete cover but also the outer permanent casing. Secondly, leaving it in place avoids the construction difficulties in removing the casing that could damage the concrete. Thirdly, there can be irregularities in the surface texture of the concrete after removal of casing due to small bleed channels or other surficial imperfections that require additional surface preparation. Any perceived aesthetic benefits of exposed concrete compared to steel casing can often be mitigated with coatings on the casing, or by other means.

An example of a removable casing is shown in Figure 5-17; this photo is taken from the I-95 Fuller Warren Bridge over the St. Johns River in Jacksonville, Florida. For this project, the removable casing was fabricated with a split seam that extended the entire length of the casing and was joined by a mechanical pin arrangement that kept the joint closed during casing installation and concrete placement,

and then expanded to facilitate removal of the casing after the concrete achieved the required strength. A rubber gasket was placed in the joint in an effort to make the joint water tight. In this example, the 72-inch diameter casings were advanced with a vibratory hammer through soft river bottom deposits either to a stiff silty clay layer or to limestone. After the drilled shaft concrete set, the pin mechanism was lifted to expand the joint, making the inside diameter of the casing slightly larger than the diameter of the drilled shaft, and allowing the casing to be lifted off the drilled shaft. The contractor selected this method to allow re-use of the casings for a number of offshore foundations, and thereby reduce the cost of steel casing. However, the use of removable casing for this project presented several problems that are often encountered with this type of solution:

a. After the initial use of the casing, the joint was typically not water tight despite cleaning and repair of the joint,

b. The contractor had difficulty opening the split joint, possibly due to fouling of the mechanism with concrete,

c. Once the joint was opened, the contractor had difficulty lifting the casing off the drilled shaft even with the use of a vibratory hammer,

d. When the casing was removed, diver inspection identified surface defects on the drilled shaft, including washout of cement along portions of the drilled shaft that had been adjacent to the split joint, numerous spalls and bleed water cavities around the remainder of the drilled shaft, and locally exposed steel reinforcement, and

e. To correct the observed defects, costly underwater remediation measures had to be implemented.

As this project case history illustrates, the use of removable casing may pose risk of structural defects to the drilled shafts. In addition, inspection of the completed drilled shafts and repair of any identified defects is complicated since this work must be accomplished under water, sometimes working under difficult conditions of limited visibility and swift currents. Accordingly, the use of removable casing at offshore foundations should generally be avoided.

Where it is specified to remove portions of exposed permanent casing, removal should generally be limited to the section of the drilled shaft above water level. In such cases, the removal would typically be accomplished by torch cutting the steel into sections, taking care to avoid damaging the underlying concrete surface, and detaching the individual sections from the surface of the concrete. Any concrete defects exposed after removing the casing segments can then be repaired using appropriate methods. If the exposed concrete is entirely above the water level, such repairs can be accomplished with less effort and with greater reliability.



Figure 5-17 Locking Mechanism for a Removable Casing

An alternative approach that may entail less risk of defects in the shaft is illustrated in Figure 5-18. This approach uses a temporary casing which is sufficiently large to function as a cofferdam. The drilled shaft constructed through the temporary casing may include a permanent casing or may simply be constructed using a drilling fluid in an uncased hole. The concrete placement can be terminated below the water surface and a removable form placed inside to form the column and splice the column reinforcement to the drilled shaft reinforcement. With this solution, the removable form is not subject to the handling stresses of a temporary casing, and the concrete within the form can be placed in the dry after removal of laitance at the cold joint. After removal of the column form, the temporary casing extending above the top of shaft cutoff can be removed with torches, using divers for casing removal below water level.

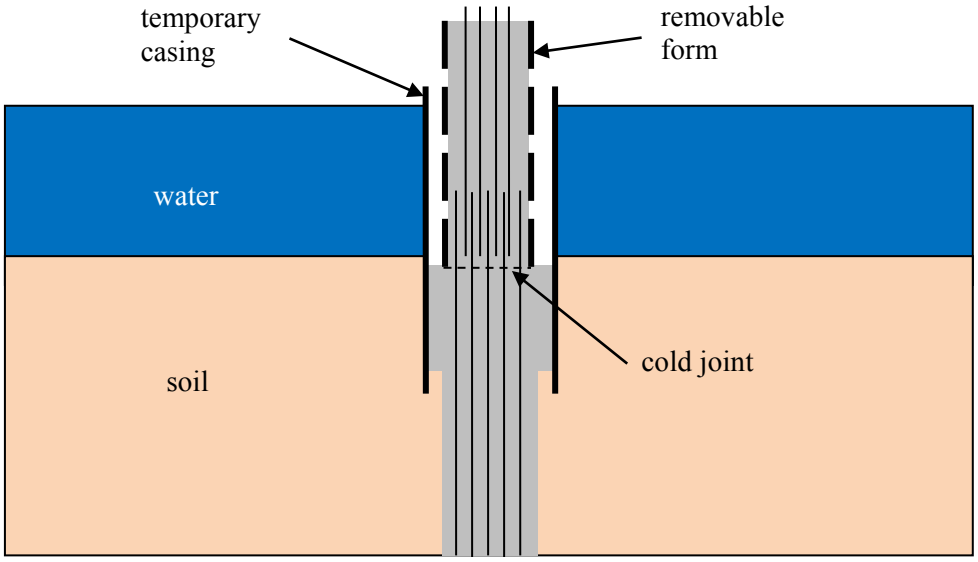


Figure 5-18 Construction Joint Below the Water Surface

5.3 PERMANENT CASING

As implied by its name, permanent casing remains and becomes a permanent part of the foundation. An example of the use of permanent casing is when a drilled shaft is to be installed through water and the protruding portion of the casing is used as a form. A possible technique that has been used successfully is to set a template for positioning the drilled shaft, to set a permanent casing through the template with its top above the water and with its base set an appropriate depth below the mudline, to make the excavation with the use of drilling slurry, and to place the concrete through a tremie to the top of the casing.

One consideration for using permanent casing is the time that will be required to place the concrete for a deep, large-diameter, high-capacity drilled shaft founded in sound rock. Control of the concrete supply may be such that several hours could pass between placing the first concrete and extracting temporary casing. In that case, the concrete may already be taking its initial set when the seal is broken by raising the casing, making it difficult to extract the temporary casing without damaging the concrete in the drilled shaft. In such as case, permanent casing may be specified.

Another common situation for using permanent casing is when the drilled shaft must pass through a cavity, as in a karst formation. The permanent casing becomes a form that prevents the concrete from flowing into the cavity. In addition to the cost of the additional concrete lost due to exterior voids in the rock, the flow of concrete into large cavities can result in mixing of soil or water into the drilled shaft, producing a void in the structure. It can also lead to distortion of the steel reinforcement cage.

Permanent casing is also commonly used for drilled shafts that extend through very soft soils, such as marsh deposits, to reach an underlying stratum which is more stable. In such cases, the permanent casing is used to prevent the outward bulging of the fluid concrete into the surrounding soft soils. If a bulge forms at an elevation corresponding to an extremely soft stratum, there can be a risk of defects in the concrete due to a neck in the shaft above the bulge, or deformation of the reinforcement cage.

5.3.1 Types and Dimensions

The types and dimensions of permanent steel casing are similar to those described previously for temporary casing. The major difference is that the permanent steel casing can be installed in longer sections of pipe and may be driven into place (similar to the installation of a steel pipe pile) since it does not need to be extracted. If the permanent casing is to be used as a structural component within the drilled shaft, the casing dimensions, material properties, and welds are typically shown in the contract documents and are subject to quality control and documentation as would be required for a steel pipe pile or any steel structure.

The left photo in Figure 5-19 shows permanent casings extending into a cofferdam after placement of the seal concrete and dewatering of the cofferdam. These permanent casings were used to extend the shafts through the river water to an underlying rock bearing layer. The casings were also designed to utilize the bond between the casing and the seal concrete to engage the axial resistance of the drilled shaft against the upward water pressure on the base of the seal; in this way, the thickness of the seal was reduced compared to the thickness of seal that would be required based on the dead weight of concrete alone. The right photo in Figure 5-19 shows the drilled shaft reinforcement after the exposed casings were removed.



Figure 5-19 Permanent Casing Used for a Shaft Group Foundation in a River

Some additional types of materials might be used for permanent liners, such as the corrugated metal pipe (CMP) illustrated in Figure 5-20. This material is sometimes used for a liner at shallow depth because of the relatively low cost. However, CMP is relatively flexible and cannot be subject to installation stresses as conventional thicker walled steel pipe.

A semi-rigid liner may also be used to minimize the skin friction that results from downdrag or from expansive soils. Coatings that have a low skin friction (such as bitumen) have also been used. Liners made of two concentric pressed-fiber tubes separated by a thin coating of asphalt have been found to be effective in reducing skin friction in drilled shafts constructed in expansive soils by as much as 90 per cent compared to using no liner.

Flexible liners are used infrequently in the United States, but can have an important role in certain situations. Flexible liners can consist of plastic sheets, rubber-coated membranes, or a mesh. The rebar cage can be encased in the flexible liner before being placed in a dry or dewatered hole; then, the concrete is placed with a tremie inside the liner. The procedure is designed to prevent the loss of concrete into a cavity in the side of the excavation or perhaps to prevent caving soil from falling around the rebar cage during the placement of the concrete. Flexible liners are applicable only to those cases where the drilled shaft is designed to develop the required resistance entirely below the level of the liner, because skin friction in the region of the liner cannot be computed with any accuracy.


Figure 5-20 Corrugated Metal Pipe (CMP) Used as Permanent Liner

5.3.2 Installation of Permanent Casing

Permanent steel casing may be installed using any of the methods described previously for temporary casing, or the permanent casing may be driven into place using an impact hammer. Permanent casing installed into an oversized hole may be sealed into an underlying rock formation by twisting or driving. In this case, it is often necessary to fill the annular space with tremie grout in order to provide transfer of lateral soil resistance. Filling of the annular space may be unnecessary if the overburden soil is neglected for lateral loading or subject to scour.

Installation of the casing by driving can be an effective and efficient means of installing a permanent casing, since it will not need to be extracted. Since installation of a large steel pipe using an impact hammer subjects the pipe to driving stresses, the drivability of the pipe must be considered as described in the FHWA Driven Pile Manual (FHWA-NHI-16-009 by Hannigan et al., 2016). There are obvious limitations to the ability to drive large diameter steel pipe into hard soils or rock, and boulders can be particularly troublesome. Where rock or boulders are anticipated, impact driving of permanent steel casing into these materials can result in deformation of the end of the casing so that drill tools cannot pass; in such cases a more attractive alternative may include the placement of permanent casing into a pre-drilled hole.

5.3.3 Effects of Permanent Casing on Axial and Lateral Resistance

If the soil within the cased zone is scourable or not capable of providing a significant contribution to the design, then the resistance of the soil around the permanent casing should not be considered a part of the design, and the method of installing the casing is unimportant from this perspective. If the soil within the cased zone is considered to provide a significant contribution to axial resistance, then the casing must be installed in such a way as to provide good load transfer through side resistance. Casing installed into an oversized hole generally cannot be relied upon to provide axial load transfer.

Even if there is no reliance placed on the cased zone for axial resistance, there may be other considerations related to the use of an oversized hole around the outside of a permanent casing. If lateral resistance is required within the zone of a permanent casing installed into an oversized hole, then the annular space around the outside of the casing should be filled with grout. An unfilled oversized hole can also provide an unintended seepage conduit, which could present a problem when working near flood control levees or other water retention structures, or when there is a risk of cross contamination of aquifers in areas where contaminated soils are present. Expansive soil or rock strata at depth could also be exposed to increased water content if an oversized hole allows downward migration of water alongside the permanent casing.

Casing which is driven using an impact hammer and left in place should provide similar axial side resistance to that of a driven steel pipe pile and may be considered as such. Caltrans often refers to this type of permanent cased hole as a “Cast-in-Steel-Shell” (CISS) pile. Where permanent casing is vibrated into place, the axial resistance of the casing in side shear may be less than that of an impact driven casing. Because steel bearing piles are not normally installed in this way, the normal methods of estimating axial side resistance for steel pile piles may not apply.

A permanent casing can contribute to the structural capacity and bending stiffness of the drilled shaft as discussed in Chapter 12. However, since corrosion will decrease the thickness of the steel casing with time, this should be considered in determining the contribution of the casing to structural capacity. Aggressive conditions are a particular concern for casings in contact with fill soils and low pH soils, and

those located in marine environments. Aggressive conditions are identified by determining specific properties of the fill, natural soil, and groundwater. Aggressive conditions are identified if the soil has a pH less than 4.5, or if the soil resistivity is less than 2000-ohm-cm. Chloride ion content and/or sulfate ion content should be conducted for soil resistivity values between 2000-5000 ohm-cm. Aggressive soil conditions exist if the sulfate ion content exceeds 200 parts-per-million (ppm), or the chloride content exceeds 100 ppm. Soils with resistivity greater than 5000 ohm-cm are considered non-aggressive. Hannigan et al. (2016) report a conservative estimate for a corrosion rate of 0.003 inch/year for steel piles buried in fill or disturbed natural soil. An in-depth review of corrosion is beyond the scope of this manual, and the reader is referred to FHWA-NHI-16-009 (Hannigan et al., 2016); AASHTO Standard R 27-01 (2004); and FHWA-NHI-00-043 (Elias, et al., 2001).

5.4 CONSTRUCTION USING DRILLING FLUIDS

Another means to stabilize a drilled shaft excavation is with the use of drilling fluids within the hole. Fluids can include water or water mixed with minerals (typically bentonite clay) or synthetic polymers. Drilling fluids provide stability by providing a fluid pressure within the drilled shaft excavation greater than the pressure of the groundwater to prevent destabilizing seepage into the hole, as illustrated in Figure 5-21.

Additives, such as minerals (bentonite) or polymer, are used to help contain the fluids within the hole and minimize fluid loss through seepage out through the borehole wall, thereby allowing the positive head pressure to be maintained. Water mixed with additives to alter the fluid properties is typically called "slurry" and the construction technique is sometimes referred to as "slurry drilling".

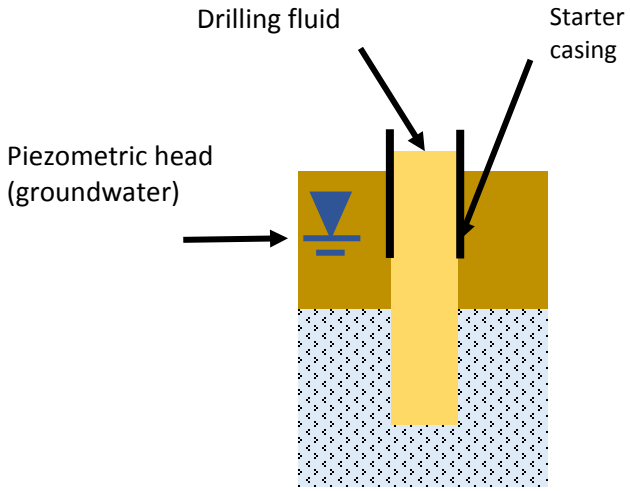
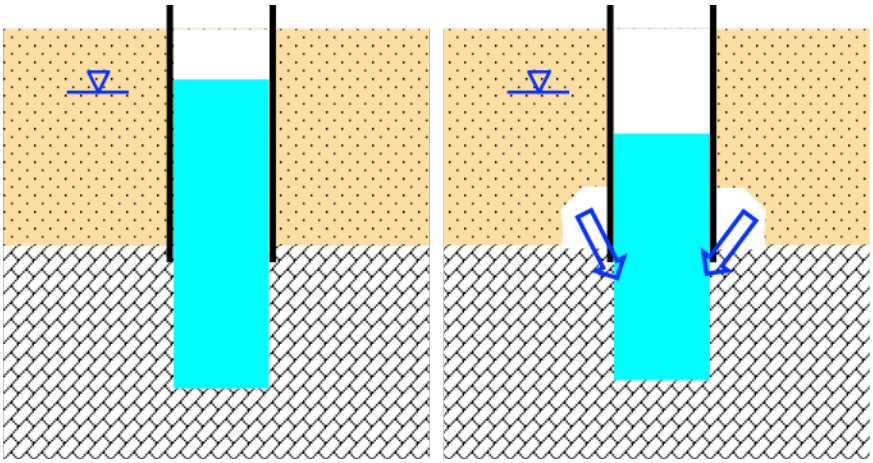


Figure 5-21 Differential Fluid Head Pressure Used to Maintain Borehole Stability

5.4.1 Water as a Drilling Fluid

Water alone is not typically suitable for use as a drilling fluid in uncemented granular soils because water flows out of the hole too fast to maintain a positive head pressure, and therefore the fluid would provide no stabilizing effect. However, if the soil has sufficient cementation or if the uncased excavation is entirely in rock, water may be utilized effectively as a drilling fluid for several purposes.

In a wet, but otherwise stable excavation through pervious rock or other strong and stable materials, the excavation may simply be filled with water in order to counter the tendency for seepage into the excavation to occur. An example would be a drilled shaft that has casing seated into rock which is not sufficiently water-tight to prevent seepage into the hole. Seepage into the hole is undesirable because it may wash fines into the hole and cause voids around the bottom of the casing which could result in ground subsidence or loss of support around the casing (Figure 5-22).



a) Acceptable Practice b) Inadequate Head

Figure 5-22 Effect of Seepage Around Imperfectly Sealed Casing Due to Lack of Positive Head Pressure

Seepage into the hole (Figure 5-23) during concrete placement can adversely affect the quality of the concrete, because the groundwater head will continue to drive water into the fluid mix until sufficient concrete head is established to overcome the groundwater head. In these situations, water should be added to the excavation to counterbalance this seepage into the hole. Maintaining a positive head pressure within the hole prior to the start of concrete placement protects the integrity of the fluid concrete mix. As a general guide, seepage into an excavation which exceeds more than one inch in 5 minutes is considered excessive and the hole should be flooded prior to concrete placement.

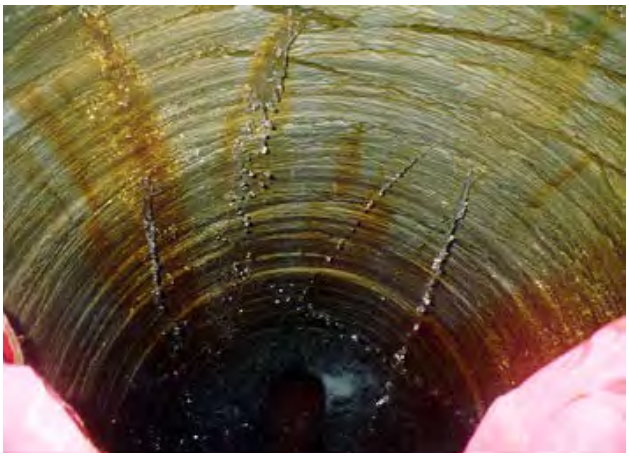
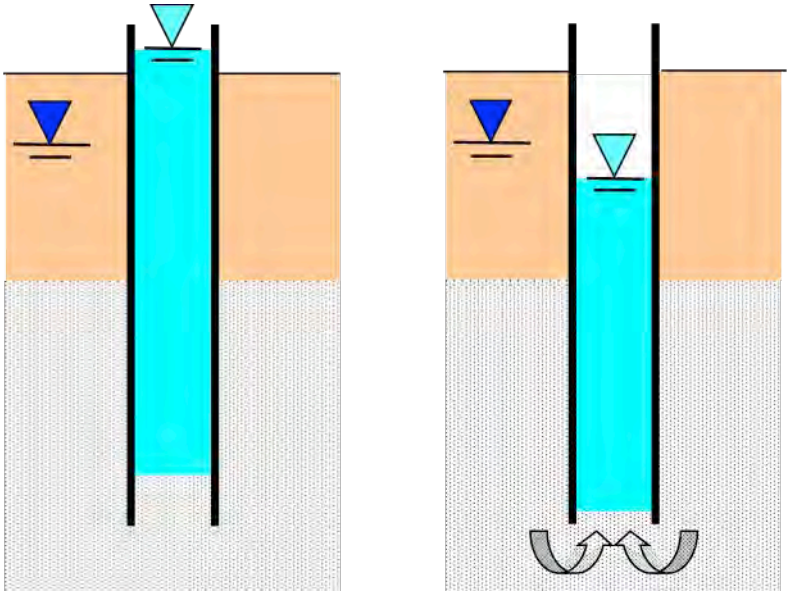


Figure 5-23 Excessive Seepage into a Drilled Shaft Rock Excavation (photo courtesy of PennDOT)

Positive water head may be needed where full length casing is employed to provide sidewall stability in permeable or unstable soils (Figure 5-24). In this situation, a positive water head is needed to avoid

upward directed flow into the base of the excavation. The upward flow could produce softening of the material at the base, or piping of cohesionless soils. In some cases, instability of the base could lead to flow of soil up and into the casing with attendant loosening of the soil around the excavation and possible subsidence at the ground surface. A positive water head within the casing, often with a plug of soil at the bottom of the casing, is used to mitigate the risk of instability at the base. More details on construction with the fully cased method is provided in Section 5.2.2.2.



a) Acceptable Practice b) Inadequate Head

Figure 5-24 Effect of Fluid Head on Bottom Stability in a Fully Cased Excavation

Water may also be used as a drilling fluid for rock excavation with tools and equipment that utilize fluids to flush cuttings from the excavation face. Examples include percussion tools, double walled core barrels, or full face rotary tools. Air may be used as the flushing mechanism in some tools, with the hole filled with water to maintain a positive head in the excavation. More details on construction with these types of tools is available in Chapter 4.

5.4.2 Mineral Drilling Fluid

Bentonite clay is the most commonly used mineral additive for drilling fluid, and is widely used in oilfield drilling applications. Bentonite is a clay composed primarily of montmorillonite clay minerals which can absorb water to many times its own weight. When added to water, relatively small amounts of bentonite form a colloidal mixture (referred to as a bentonite slurry) with the effect of increasing the viscosity of the fluid over that of water, along with a small increase in unit weight. The resulting fluid has an appearance similar to that of chocolate milk, as seen in the photo in Figure 5-25.



Figure 5-25 Mineral Based Drilling Fluid (Bentonite Slurry)

Much of the commercial bentonite used in the construction industry in North America comes from Wyoming, the name bentonite having derived from the Benton Shale there. Bentonite is a natural material composed of clays with a high proportion of montmorillonite. The desirable bentonite for construction is referred to as a high grade sodium bentonite because the clay contains a high concentration of sodium ions. Compared to other types of clay (calcium bentonite, for example), the sodium bentonite hydrates a larger volume of water in proportion to its own weight and therefore the fluid contains a relatively small volume of suspended solids at the time of introduction.

The other significant property of a bentonite slurry is that some of the minerals are filtered out at the borehole wall as the fluid passes into the soil, thereby forming a "filter cake" that reduces the permeability of the perimeter soils and thereby helps to contain the fluid (Figure 5-26). This filter cake formation is the main difference between the performance of bentonite slurry and other commonly used drilling fluids in the construction industry. The filter cake greatly improves the ability of the fluid to maintain stability of the excavation during construction, but can also adversely affect the bond between the concrete and the soil at the interface.

A key factor in the successful use of bentonite slurry is that a positive fluid head be maintained above the level of the surrounding groundwater, so that the stabilizing fluid pressure supports the sidewall of the excavation. The stabilizing pressure at any given depth is then related to the combination of head differential and unit weight as illustrated in Figure 5-27. A typical unit weight of bentonite slurry is in the range of 65 to 70 pcf, compared to 62.4 pcf for fresh water.

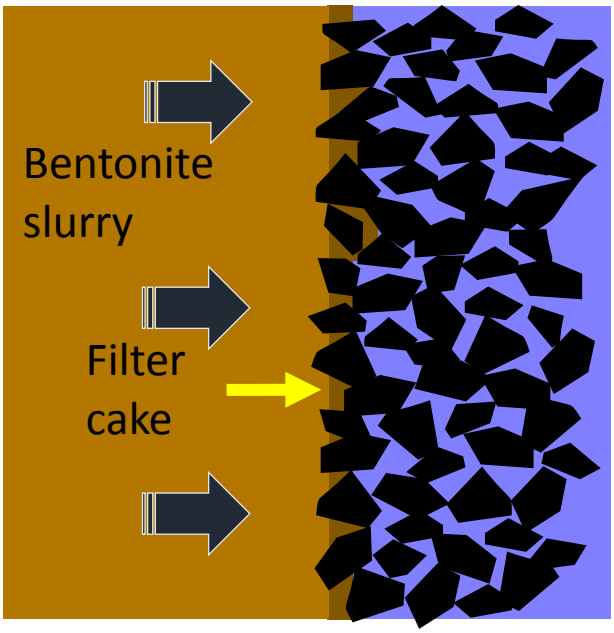


Figure 5-26 Filter Cake Formation at Borehole Sidewall with Mineral Slurry

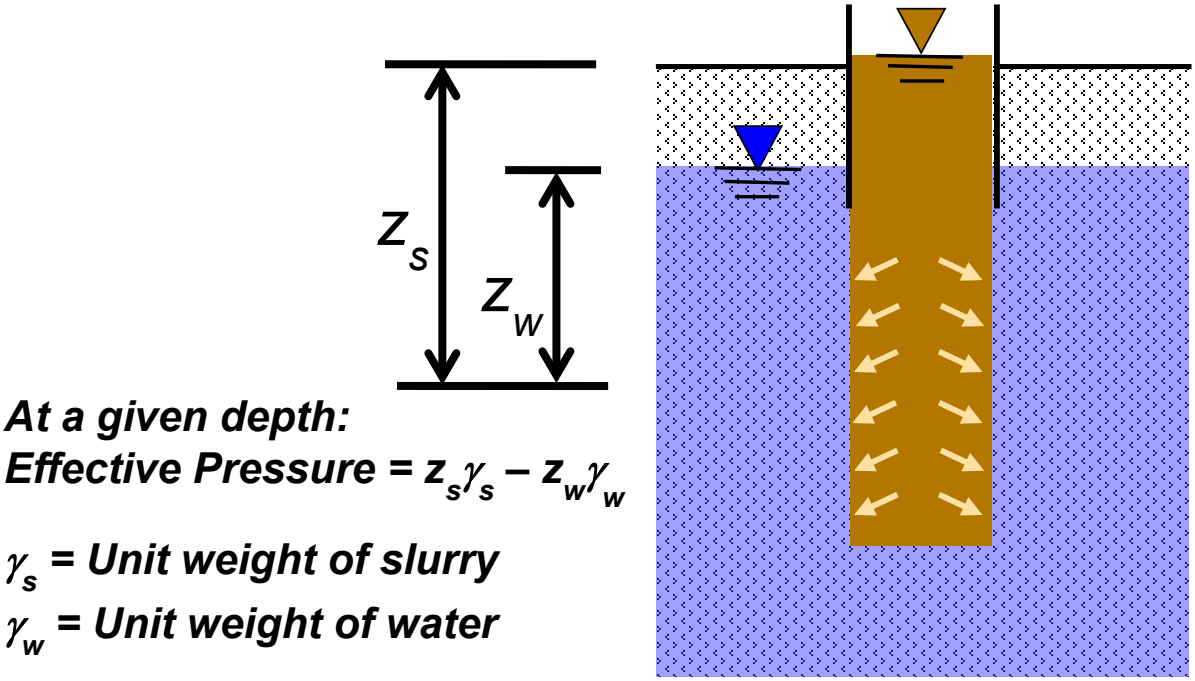


Figure 5-27 Effective stabilizing pressure provided with bentonite slurry

Although a 10-ft head differential between support fluid and groundwater is generally considered desirable, with bentonite slurry a 5-ft minimum head differential is often sufficient because of the additional benefit of the unit weight difference. The filter cake formation at the borehole wall is also beneficial in that the pressure gradient occurs over a very short distance at the very face of the excavation.

Prior to introduction into the excavation, bentonite slurry must be thoroughly mixed with vigorous shearing, and a sufficient period of time is needed for hydration of the bentonite in water (typically 24 hrs minimum). A typical mixing plant setup is illustrated in the photo of Figure 5-28. This process also minimizes the unmixed clay in the suspension which could either contaminate the concrete or lead to excessive buildup of thick filter cake at the borehole wall.



Figure 5-28 Mixing plant and holding tanks for bentonite slurry preparation

Because of the concern about potential detrimental effects of excess filter cake thickness, the exposure time of the excavation should be limited prior to placement of concrete. This may not be much of an issue for smaller drilled shafts in soil where excavation and concrete placement can be completed in the same day, but can be a consideration for larger diameter or deeper holes that are left open under bentonite slurry overnight. If the excavation cannot be completed and concrete placement commenced within a few hours, it may be necessary to take active measures to remove filter cake buildup with a special tool such as a bucket equipped with a sidewall wire brush or an auger with protruding teeth around the perimeter, as discussed in Chapter 4. The final cleanout of the hole must then be completed, rebar placed, and concrete placement begun in a timely manner. Bentonite materials can also adversely affect axial resistance for circumstances in which drilled shafts are advanced into rock and the side resistance in a rock socket is an important part of the design.

When the pore sizes in the formation being excavated are large (as in gravelly soils or poorly graded coarse sands) the filter cake may not form as effectively at the borehole wall. Bentonite slurry may still be effective for restricting fluid loss and providing stability in gravel if the gravel contains sufficient fines and the fluid has sufficient gel strength. If the bentonitic slurry proves ineffective, special techniques (for example, use of casings, additives or other types of drilling fluids, or grouting of the formation) may be required to stabilize the borehole.

Control of the unit weight and suspended solids content within the slurry is required for successful completion of a drilled shaft. After mixing, mineral slurries have unit weights that are slightly higher than the unit weight of the mixing water, with a specific gravity typically about 1.03 to 1.05 after initial mixing. During excavation, particles of the soil or rock being excavated will be mixed into the slurry and become suspended. Below a certain concentration, the soil particles will stay in suspension long enough for the slurry to be pumped out of the borehole and/or for the slurry (with suspended cuttings) to be completely displaced by an upward flowing column of high-slump fluid concrete. However, as drilling progresses and the slurry picks up more soil, its unit weight and viscosity will increase.

The slurry must be cleaned prior to concrete placement because excessive suspended soils can settle out, either onto the base of the excavation after the bottom of the hole is completed and inspected. Excess suspended soils can also tend to settle onto the fluid concrete during concrete placement, leading to entrapped pockets of sand within the completed drilled shaft. A small amount of suspended material will generally remain in suspension during concrete placement, particularly for a smaller drilled shaft where the concrete placement time may only take an hour or less. For a large diameter and deep drilled shafts, where concrete placement may require several hours, the slurry must be cleaned to a greater degree.

Suspended sand particles in a mineral slurry can be removed by processing the fluid through a de-sanding unit as shown in Figure 5-29. The fluid is pumped from the base of the excavation using an air-lift or hydraulic pump (the bottom of the fluid column will contain the highest amount of sediments) and circulated through this plant while fresh, clean slurry is added at the top of the excavation. The plant shown in Figure 5-29 contains cyclones and screens that are capable of removing sand-sized and larger particles. After processing the fluid in this manner, the slurry is checked for sand content, density, and viscosity to ensure that the fluid in the completed hole is ready for concrete placement.



Figure 5-29 De-sanding plant for bentonite slurry

Silt and clay sized particles can be removed using centrifuge equipment, but normally if the slurry becomes too heavy or viscous in spite of a low sand content it is just discarded and replaced with fresh fluid.

Groundwater that has a high salt content may cause flocculation and failure of the particles to remain in suspension. Bentonite can sometimes be used for limited periods of time in saline water by first mixing it with fresh water and then mixing the resulting fluid with additives such as potassium acetate to impede the migration of salt into the hydrated zone around the clay plates, sometimes referred to as the "diffuse double layer." With time, however, the salts in salt water will slowly attack the bentonite and cause it to begin to flocculate and settle out of suspension. Therefore, in this application, careful observation of the slurry for signs of flocculation (attraction of many bentonite particles into clumps) should be made continuously, and the contractor should be prepared to exchange the used slurry for conditioned slurry as necessary.

Minerals other than bentonite are used in limited amounts under certain circumstances. The most common are the minerals attapulgite and sepiolite. Typically, these are used for drilling in permeable soils in saline environments at sites near the sources of the minerals (*e.g.*, Georgia, Florida, and Nevada), where transportation costs are relatively 1ow. Unlike bentonite, attapulgite and sepiolite are not hydrated by water and therefore do not tend to flocculate in saline environments. These minerals do not tend to stay in suspension as long as bentonite, and require very vigorous mixing and continual remixing to place and keep the clay in suspension. However, since hydration is not a factor, the slurries can be added to the borehole as soon as mixing is complete. They do not form solid filter cakes, as does bentonite, but they do tend to form relatively soft, thick zones of clay on the borehole wall, which are generally effective at controlling filtration and which appear to be relatively easy to scour off the sides of the borehole with the rising column of concrete. It should always be verified by testing or experience that the mineral selected for slurry is compatible with the groundwater chemistry, especially at sites with low pH or contamination.

Because of turbidity issues with bentonite slurry and the relative inability to remove bentonite once it is mixed, the disposal of bentonite can be a significant cost and/or environmental consideration in some areas.

5.4.3 Synthetic Drilling Fluid

In the last 20 years, synthetic (polymer) drilling fluids have replaced bentonite slurry on most drilled shaft applications in North America, although the prevalence varies locally. The use of polymer drilling materials worldwide has also increased dramatically. Polymers function in a different way than bentonite, with some advantages and some limitations as discussed below.

The type of synthetic polymers used in drilling slurry are long chain-like hydrocarbon molecules which interact with each other, with the soil, and with the water to effectively increase the viscosity of the fluid. The appearance of the polymer fluid is that of a slippery, slimy, viscous liquid as is evident from the polymer fluid dripping off the tool in the photo in Figure 5-30. A scanning electron micro-photograph of a polymer slurry magnified to 800 times its actual size is shown in Figure 5-31(a). The polymeric strands form a three-dimensional lattice or web-like structure that can form a membrane (Figure 5-31(b)) on the excavation sidewall if a positive fluid head is maintained. This membrane can be noticeable in some cases when a drill tool excavates a clump of soil that has a sticky, wetted surface but appears to have little penetration of fluid into the mass.

Although there may be some indication of a polymer membrane at the soil interface, there is no formation of a filter cake as with bentonite. Without a low-permeability filter cake, polymers may have a greater tendency to lose fluid into the soil around the excavation with time compared to bentonite. However, this lack of filter cake provides benefit in terms of the side resistance at the concrete/soil interface, since the polymers fluids that are in widespread use have not exhibited the detrimental effect on concrete/soil bond that is associated with bentonite filter cake buildup.



Figure 5-30 Synthetic (polymer) Drilling Fluid



a) Scanning Electron Micro-Photograph
Photo Courtesy of University of Missouri-Columbia

b) Polymer Membrane
Photo Courtesy of Ken Goodhue

Figure 5-31 Photographs of Polymer Fluid

Polymers are delivered in either liquid or dry granular form and are mixed and hydrated prior to introduction into the excavation (Figure 5-32). The amount of polymer required to prepare the slurry is generally much smaller compared with the quantity of bentonite clay to prepare a similar volume of bentonite slurry. The mixing includes agitation and circulation to disperse the polymer, but the shearing action that occurs with some types of pumps (beneficial to bentonite) can break down the long chain polymer molecules and is therefore avoided. For similar reasons, polymers are not typically used with circulation drilling or with hydromill equipment for diaphragm walls or barrette construction (see sections 4.2.3.6 and 4.2.4), because the continuous pumping tends to break down the polymer.



Figure 5-32 Mixing Polymer Based Drilling Fluid

Since the polymers add little weight to the fluid, the unit weight of the drilling fluid is not much greater than that of the water used to prepare it. Given the low unit weight, lack of filter cake and potential fluid loss, the positive fluid head is critically important to achieve a stabilizing effect on the excavation with polymer slurry.

The chemistry of polymer fluids is such that the fluid tends not to incorporate dispersive clays into the fluid and thereby reduces the tendency of native soil or rock particles to become part of the slurry. This effect can also help to preserve the integrity of the rock socket into clay-shale formations which may be prone to rapid weathering upon exposure in a borehole (Axtell et al., 2009).

Another notable difference compared to bentonite is the fact that the polymer fluids do not hold soils in suspension; consequently, settlement of even fine grained soils such as silt can occur after completion of the excavation if the slurry is not adequately cleaned. The de-sanding plant used for cleaning bentonite is not suitable for polymer because the polymer tends to clog the screens, and the shearing action of the equipment tends to break down the polymer. The typical method for cleaning a polymer is to add flocculating agents to help drop suspended solids out of suspension and then provide quiet time for the sediments in the fluid to settle out. A good practice is to fully exchange the drilling fluid with fresh slurry by pumping from the base of the excavation to holding tanks (Figure 5-33) where the sedimentation can take place.

After completion of the work, the polymers can be broken down with a de-activating agent (bleach works on most types of polymers), causing the suspended solids to drop out quite easily. This property provides one of the attractions with polymer slurry in that disposal can often be accomplished with relatively little cost or effort compared to bentonite.



Figure 5-33 Sedimentation Tanks for Polymer Drilling Fluid

A comparison of the polymer drilling fluids to bentonite suggests that there are advantages and limitations of each, as indicated in the table below.

Polymer	Bentonite
Easy to mix, less time to hydrate	Mixing, hydration more involved
Time required for de-sanding, removal of fines	De-sanding equipment, handling more efficient
No filter cake – greater fluid loss and less stabilization of the hole	Filter cake + weight improves stabilization of hole, especially in coarse granular soils
No filter cake – less detrimental impact on side resistance	Filter cake can affect side resistance; exposure time should be limited; additional preparation may be needed
Polymers can break down due to shearing, pumping, but easy to dispose	Generally more costly to dispose, but easier to reuse

5.4.4 Mineral/Polymer Blended Drilling Fluid

Blended slurries consist of mixtures of minerals (generally bentonite) and polymers. In some situations, blended slurries can potentially be designed and used in a manner that takes advantage of the beneficial

characteristics of each. For example, the inclusion of polymers within a bentonite drilling fluid can be effective in minimizing the filter cake thickness and can also reduce the tendency of the fluid to incorporate native clays (and increase density, viscosity, etc.) during drilling.

The use of blended drilling fluids represents a specialty field that requires expertise beyond what is normally available on most drilled shaft projects, and there is relatively little experience in U.S. practice with blended slurries. Specifications developed for mineral slurries or commercially available polymer slurries likely will not be suitable for blended slurries. Blending is not recommended unless those involved have the knowledge and experience to determine appropriate specifications and quality control/quality assurance procedures for its use, given the site-specific ground conditions.

Blended bentonite and polymers are also available as packaged products that are marketed as "extended" bentonites. The polymer additive reduces the quantity of bentonite needed to produce a given amount of slurry, which is an economic consideration, since high-quality bentonite is becoming harder to find. However, since the properties of extended bentonites can be affected significantly by the type of polymer used, it is important for the end user (contractor) to work closely with the bentonite supplier to understand the composition and the behavior of the resulting slurry.

5.4.5 Quality Control and Inspection

Cleaning the excavation

Both the base of the excavation and the drilling fluid itself should be reasonably clean and free of debris. If excessive amounts of cuttings or other debris are trapped on the bottom of the hole, the base resistance of the completed drilled shaft could be affected by these compressible materials. Even if the base resistance is relatively unimportant for design, an excessive amount of debris could become mixed with the concrete during tremie placement, leading to possible defects in the concrete. If the fluid contained excessive suspended solids that could settle out during tremie placement, this material could also contaminate the concrete. So it is important that the fluid-filled excavation be cleaned prior to concrete placement.

It is important to understand that it is not physically possible or necessary to have perfect cleaning of the base of the excavation; a small amount of material left on the base typically has little or no consequence to the performance of the completed foundation. A typical specification (Chapter 14) requires that no more than three inches of sediment or loose or disturbed material may be present just prior to concrete placement to avoid concrete contamination, and that 50 percent or more of the shaft area should have no more than 1/2 inch where base resistance in rock or strong material is considered for design. Some agencies limit the maximum thickness to 1-1/2 inches for drilled shafts that rely on base resistance for a large portion of the required axial resistance.

It may be possible to remove debris simply by using a flat-bottomed cleanout bucket as shown in Figure 5-34, especially if the hole has had a quiet period to allow any suspended materials to settle out. Depending on the depth of the hole and type of drilling fluid, settlement of most suspended solids may occur overnight if the fluid is not agitated, and then a few passes of the cleanout bucket may successfully remove most materials. More photos and information on this type of tool are provided in Chapter 4.



Figure 5-34 Flat Bottomed Cleanout Buckets

Even after waiting for suspended solids to settle to the bottom, if many passes of a bucket are needed to remove the material this process may stir the sediments up again. A more effective tool for cleaning the fluid is a pump like the airlift or hydraulic pumps shown in Figure 5-35 and Figure 5-36, which lift fluid that is laden with sediment from the base of the excavation while the hole is replenished with clean fluid at the top. Airlift pumps work by injecting air into the bottom of the fluid-filled pipe, and the buoyancy within the pipe begins the circulation process removing fluid from within the drilled shaft excavation. The action of the hydraulically operation mechanical pump can be controlled more easily than an airlift; sometimes an airlift can pump so fast that it is a challenge to keep the hole recharged and avoid losing the positive head pressure that is essential to stability. A hydraulic pump can be operated at a speed more compatible with the recharge at the top of the hole.

Following completion of excavation and bottom cleaning operations, an inspection is performed to confirm that the base of the excavation has been cleaned sufficiently to meet the requirements of the project. Criteria typically specified for bottom cleanliness are noted in the Guide Specifications presented in Appendix D, and methods commonly used for inspection of bottom conditions are discussed in Chapter 15.



Figure 5-35 Airlift Pump for Cleaning Bottom and Pumping Slurry



Figure 5-36 Hydraulic Pump for Cleaning Bottom and Pumping Slurry

The photo in Figure 5-37 shows the base of a 6-ft diameter drilled shaft constructed under slurry that was exhumed as a part of a research project into drilled shaft concrete. This lowermost piece of the concrete was cut using a wire saw and stood upside down, so that the top of the concrete provides a casting in concrete of the bottom of the drilled shaft excavation; the pattern on the concrete represents the impression left by the cleanout bucket on the bottom of the hole after cleaning. This photo is evidence that the base of a drilled shaft constructed with slurry can be cleaned effectively by using the correct equipment and techniques.



Figure 5-37 Photo of the Base of an Exhumed Drilled Shaft, Cast Under Drilling Fluid

Sampling and testing the drilling fluid

Sampling and testing the drilling fluid helps to ensure that the characteristics of the fluid are within generally accepted guidelines and project specifications. This operation is most important just prior to concrete placement, but it is also important that the slurry properties be appropriate for maintaining stability during excavation. The slurry may be sampled for testing and evaluation using a device such as the one shown in the photo in Figure 5-38(a). Samples may be obtained throughout the fluid column, and tests of the fluid near the bottom are important because that's where the fluid is likely to have the most sediment in suspension.

The viscosity of the fluid is an important property for stability and to avoid fluid loss during excavation. Viscosity is not typically measured directly, but rather by using a simple standardized test to provide a measurement that reflects the fluid viscosity. The Marsh funnel shown in Figure 5-38(b) is a standard size funnel that is used to measure the time required for a certain volume of slurry to pass. The more viscous the fluid, the longer this will take. Water takes about 26 seconds for a quart to pass, and a typical time for bentonite drilling fluid is in the 30 to 45 second range. Many project specifications allow 28 to 50 seconds, but that can depend on the application. Higher viscosity would be better for stability of the excavation, but the filter cake for bentonite could be so thick as to be detrimental to the side resistance. Polymers are often used with higher viscosity, in the range of 32 to 135 seconds, partly because the lack of filter cake may require higher viscosity to limit fluid loss, and partly because the lack of filter cake eliminates the concern about detrimental effects from it.



a) Slurry Sampler



b) Marsh Funnel Test

Figure 5-38 Sampling and Testing of Drilling Fluids

Another simple test is the mud balance to measure unit weight of the fluid. The device and scale shown in Figure 5-39(a) is calibrated to indicate the unit weight for a specific volume of fluid held in the little cup on the right. Bentonite fluid with excessive density is likely to be too contaminated with soils for concrete placement, even though a heavy slurry may be beneficial for stability. Density is not normally an issue for polymer, which does not typically hold a lot of material in suspension.



a) Mud Balance for Density Testing



b) Sand Content Test

Figure 5-39 Testing of Drilling Fluids

Sand content, using a device as shown in Figure 5-39(b), gives a direct measure of the suspended particles, at least those that will not pass a #200 sieve. Although the sand content may vary during excavation, the upper limits prior to concrete placement of 4% for bentonite and 1% for polymer are typical guidelines that are routinely incorporated into project specifications. It may be necessary to reduce the sand content well below these limits for large diameter or deep drilled shafts because the extended time required for concrete placement presents greater exposure to potential sedimentation during this operation. For example, concrete placement in a 3-ft diameter shaft that is 40 ft deep may take only 15 to 20 minutes with relatively little time for settlement of suspended solids. However, concrete placement in an 8-ft diameter shaft that is 150 ft deep may take several hours, allowing more time for suspended solids to settle out during concrete placement. It is recommended that more restrictive sand content specifications be considered for any case in which concrete placement operations under slurry can potentially extend beyond two hours.

Hardness, as indicated by the pH, is important for the bentonite or polymer to work properly, and so this is normally checked at the beginning with the make-up water. pH is particularly important for polymers since the chemical reactions which cause the polymers to form is sensitive to the pH. Soda ash is typically used to increase the pH.

A device called a filter-press may be used for testing the tendency of the drilling fluid to form a filter cake. Although not a routine inspection test, this test provides a relative indication that is useful for adjusting the fluid properties so as to control the filter cake thickness with bentonite slurry. The device consists of a small slurry reservoir that is installed in a frame, a filtration device, a system for collecting and measuring a quantity of free water, and a pressure source. The test is performed by forcing slurry through a piece of filter paper under a pressure of 100 psi for a period of 30 minutes. The free water that is recovered is measured in cubic centimeters, and the thickness of the cake that is formed is measured to the nearest millimeter. Before measuring the cake thickness, any superficial slurry that is not part of the filter cake is washed away.

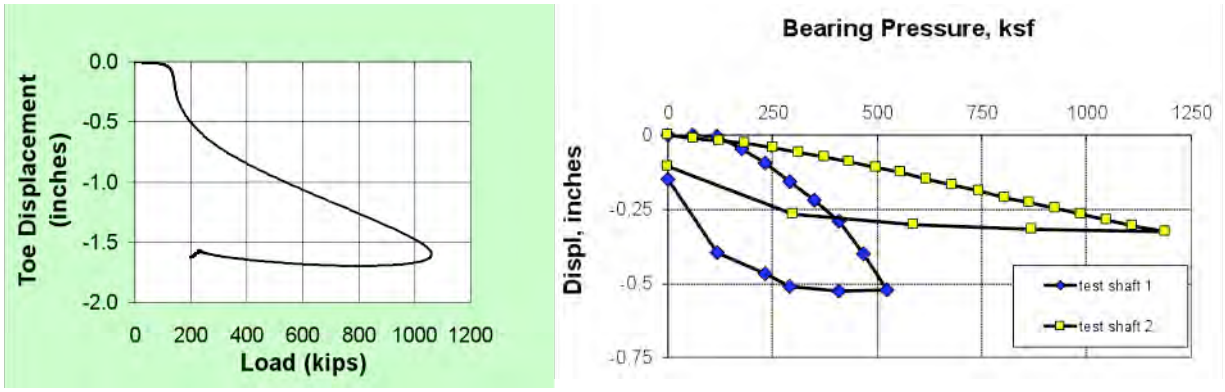
5.4.6 Effects of Drilling Fluid on Axial Resistance

The most important influence of drilling fluid on axial resistance is a positive one, by keeping a stable hole and maintaining the integrity of the bearing materials. The excavation shown in Figure 5-40 obviously needed some support, and the caving that is visible might have been avoided with the effective use of either drilling fluids or casing. If seepage into the hole occurs, even with casing, debris can flow into the excavation and be deposited onto the bottom. Uncontrolled seepage into the hole is also detrimental to concrete quality, as described previously.



Figure 5-40 Caving in an Unsupported Drilled Shaft Excavation

With the practices described in the previous section, it is often actually easier to clean the bottom of a drilled shaft under fluid than it is to clean a "dry" excavation. Use of proper drilling fluid with good base cleaning operations can avoid a "soft toe" effect that is evident in the load test results presented in the plot in Figure 5-41(a). This little "duck tail" at the beginning suggests that the bottom didn't provide much resistance on initial loading and then became stiffer after about 1/2 inch of displacement. The bottom debris effectively required some additional larger displacement to mobilize the base resistance. The plots in Figure 5-41(b) are from two load tests that illustrate the behavior of drilled shafts on rock at a site near Atlanta with a clean base. High base resistance is mobilized at very small displacements. Test shaft 1 is on weathered rock and test shaft 2 is on hard rock, but both exhibit very stiff initial resistance.



a) Load Test Results Indicating Soft Base b) Load Test Results Indicating Clean Base

Figure 5-41 Field Load Test Results Showing Effect of Base Cleanliness

Previous discussion has alluded to the fact that an excessive build-up of bentonite filter cake at the borehole wall could be detrimental to side resistance. There is increasing evidence that the unit side resistance of drilled shafts constructed in granular soil using polymer fluids is superior to that of drilled shafts constructed using bentonite, even with accepted practices. The plot in Figure 5-42 shows data from one of the earlier full scale comparison studies, which has been corroborated by other studies that show

similar evidence. This effect would not be expected and has not been observed in low-permeability cohesive soils, likely because the lack of fluid loss does not result in a filter cake deposit at the borehole wall. It is also of interest to note that the side resistance in either case appears to be mobilized at small displacements, but exhibits ductile behavior out to nearly 2 inches of displacement. It is reassuring to note that the side resistance does not typically drop away in a brittle manner; however, the magnitude is clearly influenced by the construction means and methods.

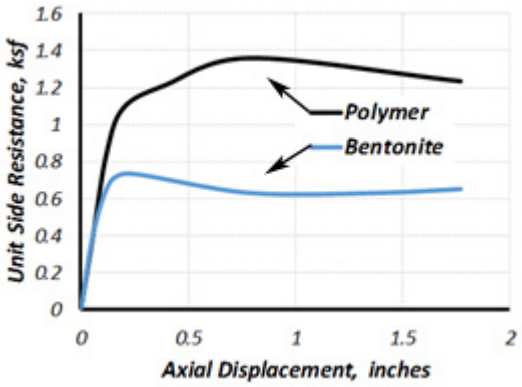


Figure 5-42 Field Load Test Comparison of Side Resistance with Different Drilling Fluids (Brown, 2002)

Another potential benefit with polymer slurry materials has been identified in recent years, and that is the tendency of the polymer to reduce the amount of "wetting" and degradation of shales at the borehole wall. Many geotechnical engineers have experience with the rapid degradation of shale exposures in road cuts, and the same thing can happen in a drilled shaft excavation. The result of degradable shale could be that the side resistance of a rock socket in such material would be affected by the degraded shale rather than the stronger intact material. Slake durability tests suggest that the polymers can reduce the tendency of some shales to degrade quickly in the presence of water (Axtell et al., 2009). Anecdotal evidence from several load tests on projects in these materials support the conclusion that polymers can be beneficial in preserving the integrity of shale bedrock bearing materials.

5.5 SUMMARY

Casing provides a variety of functions in the construction of drilled shafts, ranging from short surface casing for protecting the top of the shaft excavation, to temporary casing for supporting the hole within unstable or water bearing soil layers during shaft excavation, to permanent applications where the casing may serve as a concrete form through water or as a structural component of the completed drilled shaft, to note just a few. Whether temporary or permanent, however, the method used for installation of the casing, and for removal of temporary casing, can have a significant influence on the performance of the drilled shaft.

Drilling fluids provide a means to enhance stability during drilled shaft construction. It is therefore critically important for designers and other construction professionals to recognize and understand this technology and the role it provides in drilled shaft construction. The range of fluids can vary depending upon the application, but a consistent theme is the need to maintain a positive head pressure within the excavation at all times during excavation and concrete placement. Good practices, combined with careful quality control and quality assurance provisions, are needed to ensure that the desired foundation performance is achieved.

The information in this chapter, provides a general understanding of the construction techniques available to maintain hole stability and to facilitate installation of drilled shafts in difficult ground and groundwater conditions. This chapter provided an overview of the various applications for casings and liners, identified the common methods and equipment used for casing installation and extraction, and discussed potential effects of casings on the axial and lateral resistance of the completed shaft. Experience and research have demonstrated that drilling fluids made from both bentonite or polymers can provide a highly effective means for constructing quality drilled shafts when used properly. Factors that lead to successful performance and potential adverse impacts on performance are identified in this chapter.

CHAPTER 6
REBAR CAGES

6.1 INTRODUCTION

Design, fabrication, and installation of the reinforcing, or "rebar," cage are important steps in the drilled shaft design and construction process. Rebar cages are considered from two perspectives in this manual: (1) from a structural design perspective, including the amount and arrangement of steel necessary to resist stresses that develop in response to axial, flexural, and shear demands transmitted to the drilled shaft, and (2) characteristics of the cage from the perspective of constructability. This chapter addresses the second of these perspectives, rebar cage constructability. Structural design considerations are covered in Chapter 12.

A drilled shaft rebar cage is comprised of longitudinal bars that are normally arranged in a uniform spacing circumferentially to form a cylinder that is concentric with the drilled shaft. Transverse reinforcing is placed around and attached to the longitudinal bars. The longitudinal and transverse steel are held together by ties, clamps, rings, or welding. Other components of a rebar cage may include sizing hoops, guides for centering the cage in the borehole and for locating the tremie inside the cage, and bracing and pickup devices to aid in lifting the cage. For long cages and cages with large diameters, bracing elements should be provided to prevent permanent distortion of the cage as a result of stresses induced by lifting and placing, and for safety. Access tubes for nondestructive testing are attached to the reinforcing cage and, from a construction perspective, are part of the cage.

The amount of steel reinforcement in a drilled shaft is controlled by structural requirements. The structural designer performs analyses that typically involves applying a combination of loads (axial, lateral, and moment) to the top of the shaft, and computing the resulting stresses in the steel and concrete. The amount and arrangement of reinforcing steel is made in consideration of the stresses that will exist, using appropriate load and resistance factors in the computations and in accordance with applicable design code provisions. However, the designer often has multiple options on how the required amount of reinforcement can be provided, for example with respect to bar sizes, bundling of bars, concrete cover, location of splices, and other variables associated with detailing of the rebar cage. For drilled shafts, these design details have important implications for constructability. This chapter provides guidelines on practices for rebar cage assembly, lifting, placement and centering that have been shown to be practical, constructible and safe, and which accommodate concrete flow through the openings between bars, and discusses other issues related to drilled shaft reinforcement.

The assumption is made that the rebar cage is always placed in the excavation first, followed by concrete placement, during which the concrete flows through and around the cage. Short rebar cages may be pushed or vibrated into fresh concrete (sometimes referred to as a “wet-stick”), but such a procedure is unusual.

6.2 PROPERTIES OF REINFORCING STEEL

The American Society for Testing and Materials (ASTM) provides specifications and properties of steels that can be used in reinforced concrete. These specifications are presented in the Annual Book of ASTM Standards and are conveniently collected in Publication SP-71(08) of the American Concrete Institute (ACI, 2008). Most of the ASTM steel types and grades also have a corresponding designation from the American Association of State Highway and Transportation Officials (AASHTO), and these are specified in the

AASHTO LRFD Bridge Construction Specifications (2017b). The AASHTO LRFD Bridge Design Specifications (2017a) further define the properties of steel used for reinforced concrete. The current AASHTO design code (2017a) states:

> "The nominal yield strength shall be the minimum as specified for the grade of steel. Yield strengths in excess of 75.0 ksi up to 100 ksi may be used for design permitted by Article 5.4.3.3. . . . Bars with yield strengths less than 60 ksi shall be used only with the approval of the Owner"

Article 5.4.3.3 states:

> "Where permitted by specific articles, reinforcement with specified minimum yield strengths of less than or equal to 100 ksi may be used for all elements and connections in Seismic Zone 1."

The above code provisions therefore bracket the yield strength of steel reinforcement between 60 ksi and 100 ksi for drilled shafts supporting bridges and other structures in all seismic zones.

Properties of steel that may be employed for reinforcement in drilled shafts are shown in Table 6-1. The most widely-available and widely-used rebar steel for drilled shafts is AASHTO M 31 (ASTM A615). This product is currently available in five grades, where the grade designation corresponds to the yield stress in ksi: Grade 40, Grade 60, Grade 75, Grade 80, and Grade 100. The specifications in the table do not address welding of the M 31 and M 42 steels because these steels are not intended to be welded in normal practice. Where welding of a rebar cage is desirable, a weldable steel, such as ASTM A706, should be specified.

Rebar designated as AASHTO M18 (ASTM A1035) is a chromium alloy steel that provides a high level of corrosion resistance without the need for coating. These bars are available in grades up to 100 ksi, providing the potential for combining high strength and high corrosion resistance. Galvanized or epoxy-coated steel is also available for longitudinal and transverse reinforcement to provide corrosion resistance. Corrosion resistant rebar is sometimes specified for drilled shafts in marine environments, where the chloride content of the ground and/or surface water is high. Nicks and blemishes in the coating that may occur during lifting and placement of the cage into the excavation can become points for accelerated corrosion; accordingly, specification of coated bars can present unusual challenges for construction of drilled shafts. Alternatively, potential for corrosion can be addressed by specifying a dense concrete of low permeability, or by increasing the concrete cover.

Plain bars are not recommended and the AASHTO design code (2017a) states that "*Reinforcement shall be deformed, except that plain bars or plain wire may be used for spirals, hoops, and wire fabric.*"

Table 6-1 shows the maximum size of bar that is available for each rebar designation. The designations of deformed bars, their weights per unit length, cross-sectional areas, and perimeters are given in Table 6-2. The values shown in Table 6-2 are equivalent to those of a plain bar with the same weight per unit length as the deformed bar.

The modulus of elasticity of steel is usually taken as 29,000,000 psi. For design purposes the stress-strain curve for steel is usually assumed to be elastic-plastic, with the knee at the yield strength (Ferguson, 1981).

Table 6-1 PROPERTIES OF STEEL FOR CONCRETE REINFORCEMENT

ASTM Designation	AASHTO No.	Description	Yield Strengths ksi	Weldable	Maximum Bar Size
A615	M 31	Deformed and plain billet-steel bars	40 60 75 80 100	No	No. 18
A616	M 42	Deformed and plain rail-steel bars	50 60	No	No. 11
A706	-	Deformed low-alloy steel bars	60 80	Yes	No. 18
A1035	M18	Deformed and plain, low-carbon, chromium-alloy steel bars	60 75 100	Yes	No. 18

Table 6-2 WEIGHTS AND DIMENSIONS OF DEFORMED BARS (CUSTOMARY)

Bar No.	Weight lb/ft	Diameter in.	Cross-Sectional Area $in.^2$	Perimeter in.
3	0.376	0.375	0.11	1.178
4	0.668	0.500	0.20	1.571
5	1.043	0.625	0.31	1.963
6	1.502	0.750	0.44	2.356
7	2.044	0.875	0.60	2.749
8	2.670	1.000	0.79	3.142
9	3.400	1.128	1.00	3.544
10	4.303	1.270	1.27	3.990
11	5.313	1.410	1.56	4.430
14	7.650	1.693	2.25	5.320
18	13.600	2.257	4.00	7.090

The use of higher strength rebar, up to and including the AASHTO upper limit of 100 ksi, is becoming more common and provides both design and construction benefits. The amount of reinforcement can be reduced substantially and this benefits constructability by reducing cage congestion and reduced cage weight. Figure 6-1 shows a drilled shaft rebar cage with GR75 threaded bars that was used in the fast-track reconstruction of the collapsed I-35W bridge in Minneapolis. Threaded couplers were used to make splice connections.

A recent development (Alter, 2011; Steudlein et al., 2017) has been the use of Grade 80 and Grade 100 hollow bars as drilled shaft reinforcement. A hollow bar can be used as an access tube for crosshole sonic log (CSL) testing, eliminating the need for PVC or steel access tubes. The combination of less steel (of higher grade) and absence of separate access tubes reduces cage congestion, promotes flow of concrete, and reduces the cage weight. The hollow bars currently available meet the requirements for ASTM A615 designation. The hollow center of each bar is grouted for corrosion protection after testing is completed.



Figure 6-1 GR75 Reinforcement Cage Being Assembled, Showing Threaded Couplers

6.3 LONGITUDINAL REINFORCING

The principal role of longitudinal reinforcing steel in drilled shafts is to resist stresses due to bending, compression, and tension. The required amount of longitudinal reinforcement is determined as a function of load demand (moment, compression, and tension) and material properties, according to methods presented in Chapter 12 (Structural Design). In addition, the AASHTO (2017a) design specifications provide minimum percentages of longitudinal steel for all reinforced concrete structural members including drilled shafts. The AASHTO (2017a) design specifications explicitly require drilled shafts for bridges in Seismic Zones 3 and 4 to be reinforced over the full length of the shaft. Regardless of seismic zone, a minimum amount of longitudinal steel should be specified over the full depth of drilled shafts. As described in Chapter 12, where moment demand is minimal or non-existing (for example at great depth), and where the area of concrete in the cross section is greater than required to resist axial compression, the minimum percentage of longitudinal steel of 0.5% provides a lower bound. From a practical standpoint, full-length longitudinal reinforcement is needed to support access tubes for nondestructive testing (when specified). Also, with some methods of construction, it is desirable that the cage be able to stand on the bottom of the shaft excavation during the placement of the concrete (*e.g.* when extracting temporary casing), in which case some of the longitudinal bars must extend over the full length of the shaft.

Performance of reinforced concrete depends upon good bond between the longitudinal bars and the concrete. The surfaces of the bars must be free of excessive rust, soil, oils, or other contaminants. Deformed bars are used to ensure that adequate bond to the concrete is achieved. As the concrete rises to displace the slurry around the rebar steel, there is a possibility that water, bentonite, or polymer will be trapped around the deformations. However, there is no evidence at present to indicate that significant loss of bond occurs in wet construction if the slurry meets appropriate specifications at the time the concrete is placed.

It is possible in concept to vary the circumferential spacing of the longitudinal bars and to orient the cage in a specific direction in cases where the main forces causing bending have a known preferential direction. However, any small potential material savings that would be gained by such a procedure are generally more than offset by the risk of delays in the inspection and construction, or risks of misalignment or twisting of the cage during placement into the borehole. Therefore, the longitudinal bars are recommended to be spaced equally around the cage unless there are compelling reasons for nonsymmetrical spacing. If the number of bars in a symmetrical cage is at least six, the bending resistance is approximately equal in any direction. Figure 6-2 shows the longitudinal steel in a rebar cage being assembled on a job.



Figure 6-2 View of a Rebar Cage Being Assembled, Showing Longitudinal Steel

The minimum clear spacing between longitudinal bars, and between transverse bars such as spirals or hoops, must be sufficient to allow free passage of the fluid concrete through the cage and into the space between the cage and the borehole wall. This spacing is particularly important because drilled shaft concrete is placed without vibrating the concrete. Although the required spacing is somewhat dependent upon other characteristics of the fluid concrete mix, the size of the largest coarse aggregate in the mix is an important characteristic. The AASHTO design code (2017a) Article 5.12.9.5.2 states:

> … the clear distance between parallel longitudinal, and parallel transverse reinforcing bars in cast-in-place concrete piling shall not be less than the greater of the following:
>
> - Five times the maximum aggregate size
> - 5.0 in.

The clear spacings cited above should be considered minimum values. Research reported by Dees and Mullins (2005) suggests that a minimum spacing of eight times the size of the largest coarse aggregate in the mix is needed to avoid blocking for tremie-placed concrete. Where tremie placement of concrete is anticipated, many agencies require a minimum opening between bars which is 5 inches in both the vertical and horizontal direction, and at least ten times the size of the largest coarse aggregate in the mix. If concrete placement into a dry shaft is assured, then a smaller spacing on the order of five times the size of the largest coarse aggregate may be considered. The bar size selected for the longitudinal steel must be such that the proper clear spacing between bars is maintained. These recommendations for minimum clear spacing also apply to access tubes which may be included for non-destructive testing as described in Chapter 16.

One method of maintaining the appropriate clear spacing while providing the needed percentage of steel reinforcement is to bundle the bars in groups of two or three. Bundling of bars may require a greater development length beyond the zone of maximum moment. A cage with two-bar bundles of No. 18 bars is shown in Figure 6-3.



Figure 6-3 View of Bundled No.18 Rebar in a Drilled Shaft Cage

Some designers have used two concentric rebar cages to provide an increased amount of steel for drilled shafts with unusually large bending moments. However, the use of multiple cages significantly increases the resistance to lateral concrete flow, which greatly increases the risk of defective concrete at the perimeter of the drilled shaft and in the space between the two cages. In such cases, consideration should be given to design features that eliminate the need for multiple cages, such as using higher strength bars, bundled bars, and/or increasing the diameter of the drilled shaft.

6.4 TRANSVERSE REINFORCEMENT

From the perspective of reinforced concrete structural design, the purpose of transverse reinforcing steel is to: 1) resist shear by intersecting and impeding potential diagonal tension cracks, 2) prevent the longitudinal bars from buckling outward, thus allowing the longitudinal bars to fully develop their yield strength, and 3) provide confinement of concrete in the core of the cage to give the drilled shaft post-yield ductility. From a construction perspective, the transverse reinforcement holds the longitudinal steel in place, forming a stable structure (the cage) which can be lifted and placed as a unit into the drilled shaft excavation safely. The most common types of transverse reinforcement in drilled shafts are spirals and hoops (circular ties).

When a hoop or spiral is used, the end of the steel must be anchored to assure that the full bar capacity is achieved at the point of connection. Figure 6-4 (a) shows a plan view of what could be either a hoop or spiral, in which the bar ends have a minimum 6-inch lap and termination with a hook. ACI (2014) requires the hooks to have a 6 bar diameter extension that engages the longitudinal reinforcement and projects into the interior of the cage. This type of detail is common for columns and other above-ground members. However, for drilled shafts, protrusion of the hooks, as seen in Figure 6-4 (b), could interfere with the

introduction of a tremie or the placing of concrete by free fall. A preferred practice for drilled shafts is to lap splice the ends of the hoops or spiral. Figure 6-5 shows an example in which bundled No. 6 hoops are welded to form a splice and the ends are not hooked to the inside of the cage. An extension of the steel beyond the point where its resistance is needed ("development length") is recommended on each side of the splice. ACI (2014) recommends a development length in inches of $0.04A_b f_y/[(f'_c)^{0.5}]$ for bars of No. 11 size or smaller that take tension, such as transverse steel, where A_b is the cross sectional area of the bar in square inches, f_y is the yield strength of the steel in psi, and f'_c is the cylinder compression strength of the concrete, also in psi. Some agencies specify that spiral steel be lapped for one full turn.

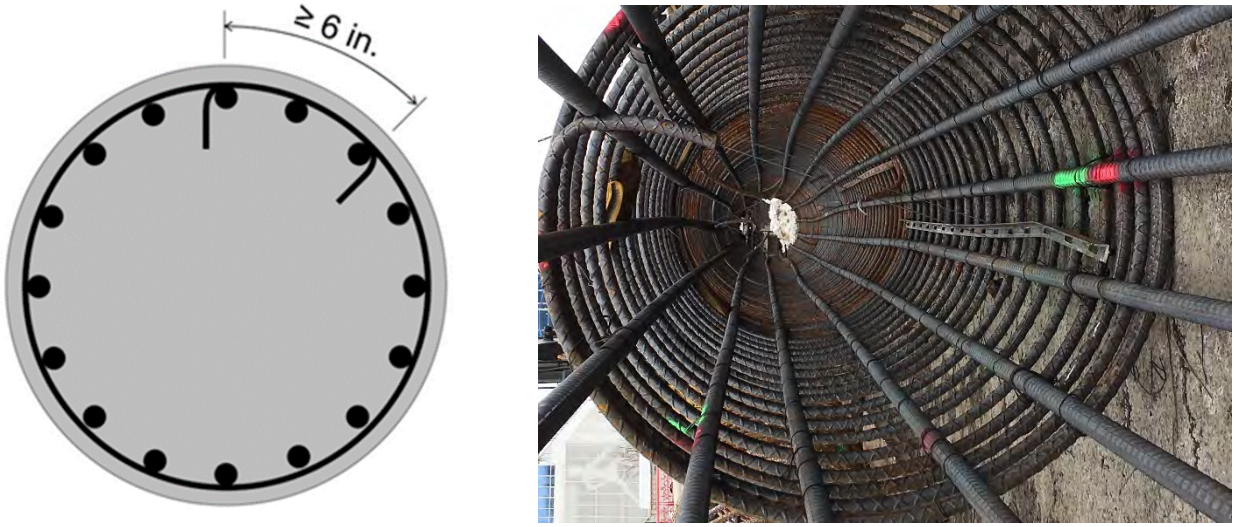


Figure 6-4 (a) Hoop or Spiral Detail with Anchored Hooks; (b) Protrusion of Hook into Center of Cage (not recommended)

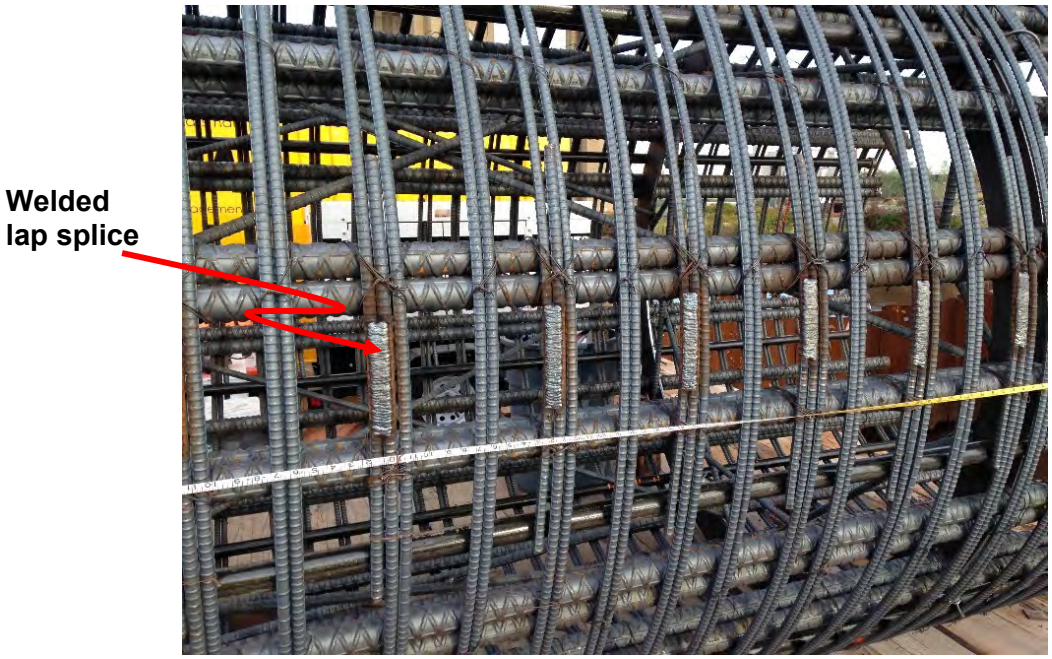


Figure 6-5 No. 6 Bundled Hoops with Welded Lap Splices.

Designs requiring ductility and core confinement in high moment regions near the top of the shaft, particularly in seismic regions, often result in relatively large amounts of transverse reinforcement. However, tight spacing on spiral reinforcement (less than 5 inch pitch) can result in constructability problems with concrete flow through the cage. The photos in Figure 6-6 illustrate problems resulting from tightly spaced spiral reinforcement and a concrete mix with insufficient passing ability for this congested condition. Failure of the concrete to flow through the cage resulted in inadequate cover and poor contact between the soil and shaft.



Figure 6-6 Examples of Inadequate Flow of Concrete through Tightly Spaced Spiral Reinforcement

Several solutions are available to address the constructability problems where a large area of transverse reinforcement is required. Use of larger bar sizes and higher strength steel can allow an increase in the bar spacing. A method used by Caltrans (Figure 6-7) is to utilize bundled hoops of larger bar sizes (up to No. 8) in order to increase the clear space between hoops to at least 5 inches. Another solution which may be appropriate in some circumstances is to utilize permanent steel casing to provide confinement and ductility near the top of the drilled shaft. Even if the casing is not considered for contribution to flexural demands, the structural benefits of including a steel liner for confinement and ductility can be substantial. Finally, if very tight spiral spacing is utilized, a concrete mix specifically designed to provide high passing ability may be used. Concrete mixes are described in Chapter 7.



Figure 6-7 Bundled Hoops Used to Improve Flow of Concrete through Transverse Reinforcement

6.5 SPLICES

Splicing of the longitudinal reinforcement is required when the length of the cage exceeds the length of the available reinforcing bars, which is normally supplied in lengths of 60 ft or less. Splices in the longitudinal steel can be made by lapping the bars so that the bond in the rebar is sufficient to develop the full capacity of the bar in tension or compression in each bar at the point of the splice. An appropriate development length, as indicated in the governing code (*e.g.*, AASHTO, 2017a) is necessary in both bars on either side of the splice. The tie wire or clamps that are used to connect the bars must have sufficient strength to allow the cage to be lifted and placed in the borehole without permanent distortion of the cage. Splices can also be welded if the appropriate steel is specified (*e.g.*, ASTM A706).

Splices in the longitudinal steel, if required, should be staggered so that all splices do not occur in the same horizontal plane along the rebar cage. Not more than 50 per cent of the splices should be at any one level. These guidelines are for constructability as well as structural considerations; if a large number of lap splices are placed at the same location, the splices can result in an obstruction to concrete flow as is the case shown in Figure 6-8.

Splices in the longitudinal steel can be made also by the use of special connectors, as illustrated in Figure 6-9. Although these various types of patented splice connectors are typically more expensive than lap splices, the use of these devices can reduce congestion in the cage. One such connector encloses the butt ends of two rebars, and the ignition of the patented material inside the connector results in a joint with considerable strength. Similar to lap splices, mechanical splices should be staggered for structural considerations.

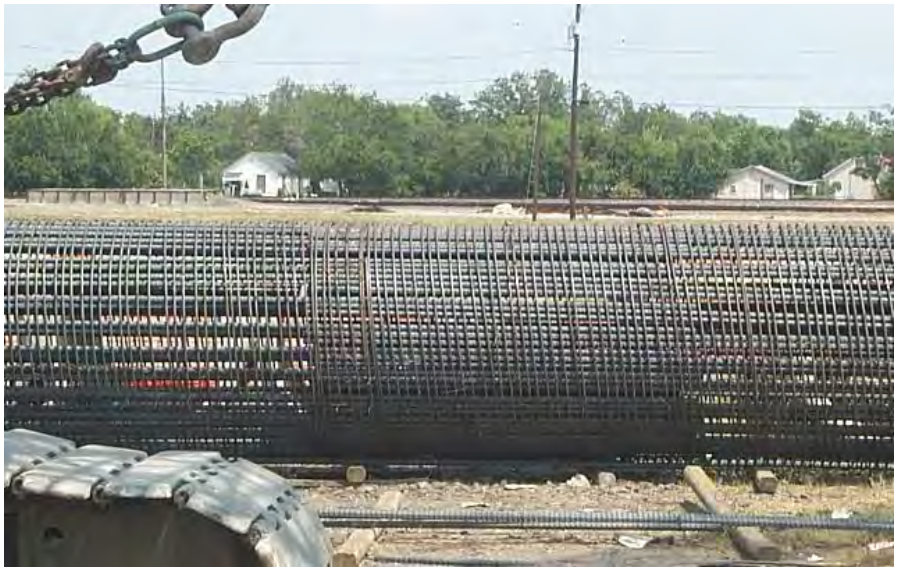


Figure 6-8 Constructability Problem from Excessive Concentration of Lap Splices

Many structural designers prefer not to place any splices in zones near the location of maximum flexural stresses in the drilled shaft-column system when large lateral loads are anticipated (as when the design includes seismic considerations). Some agencies also avoid splices in zones where the probability of steel corrosion is the highest, such as splash zones in a marine environment.

There are cases where the cage is so long that it cannot be lifted conveniently in one piece, or where restricted vertical clearance precludes installation of a full length cage. In such cases, the cage can be spliced

in the borehole subject to approval by the engineer of record. The lower portion of the cage is lifted, placed in the excavation, and held with its top at a convenient working level while the upper portion is lifted and positioned so that the two portions of the cage can be spliced together. Wire ties or clamps are usually employed to make the splices, with the ties or clamps in the longitudinal steel being staggered. The entire cage is then lowered to the correct position. Since concrete should be placed in the completed excavation as soon as possible after completion of drilling, time-consuming splicing in the hole should be minimized, or avoided if possible.



Figure 6-9 Bar Couplers Used to Construct Splices

6.6 CONNECTION BETWEEN DRILLED SHAFT AND COLUMN

Another constructability concern involves the fabrication of the connection between the drilled shaft reinforcement and the column. There are several possible approaches to the design of this connection, each of which has particular considerations in construction. A major factor relates to the tolerance in design of a splice near the top of the drilled shaft or base of the column, which may present a concern for ductility in a high moment area for seismic loading.

If the design allows a lap splice at the base of the column, a simple approach is to leave the shaft reinforcement sticking above the top of the shaft by a sufficient length to form the splice. This approach works best when the column is round and the diameter of the shaft and column reinforcing cage are of similar size. The typical minimum concrete cover on column reinforcement is around 3 inches, and the typical minimum cover for drilled shaft reinforcement is around 6 inches; accordingly, the cages will align best if the drilled shaft is specified to be 6 inches larger in diameter than the column. In addition, a planned 6-inch cover on the shaft reinforcement can allow this cage to be adjusted by 3 inches in any direction (the typical tolerance on location of the drilled shaft) so as to line up with the column cage and still maintain at least 3 inches of cover over the shaft cage. This concept is illustrated in Figure 6-10. If a minimum 6-inch cover of the drilled shaft rebar cage must be maintained, the drilled shaft diameter towards the top of the shaft can be increased using surface casing.

Another method that is occasionally used to accommodate the location tolerance of the drilled shaft and to maintain the required concrete cover for the drilled shaft rebar cage, is to design the connection at the top of the column for the same offset tolerance as the drilled shaft. This approach allows the drilled shaft rebar cage to remain centered in the drilled shaft and the column steel can be spliced directly to the drilled shaft rebar cage.

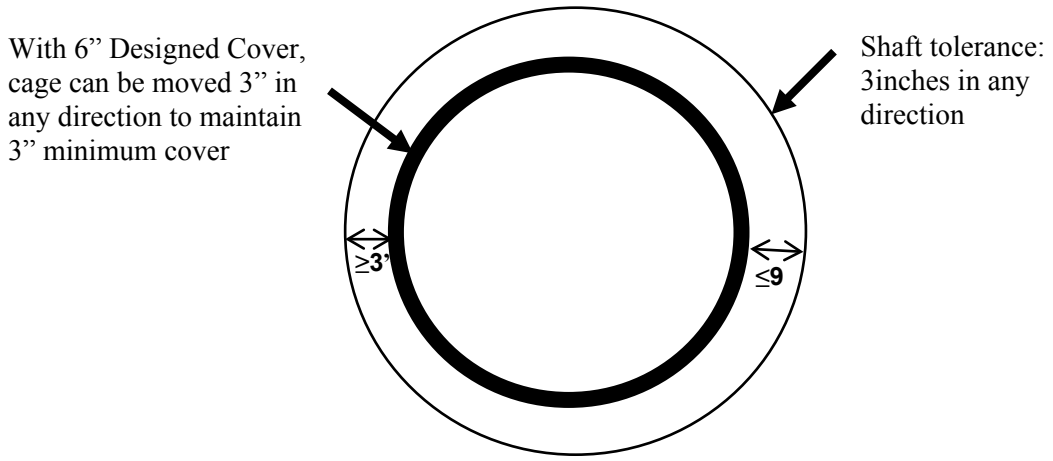


Figure 6-10 Adjustment to Drilled Shaft Reinforcement for Alignment to Column Cage

If the design requires a continuous longitudinal cage extending from the shaft into the column with no splices near the ground line (sometimes referred to as a "Type I" connection in seismic areas), then the contractor may be forced to work over and around a cage which extends many feet above the top of the shaft. This often results in a very long cage that requires special handling by the contractor. This approach will increase costs due to the need for bigger cranes to lift the taller cage and possibly extract casing high above grade. Concrete placement is also more complicated and expensive due to the projecting cage. The cage must be supported externally as the concrete is being placed and as it cures. The use of a planned 6-inch cover on the shaft reinforcement is desirable in this instance for the reasons as stated above. A Type I connection is shown in the left and center illustrations in Figure 6-11.

In some cases, the design incorporates a drilled shaft which is significantly larger than the column and is designed to have greater moment resistance so that formation of a plastic hinge from a seismic overstress condition is confined to the base of the column above grade. This approach is typically referred to as a "Type II" connection in seismic areas. A Type II connection is illustrated on the right in Figure 6-11. The normal approach is to extend the column reinforcement into the top of the shaft to form a non-contact lap splice for a sufficient distance to develop the strength of both the column and the shaft reinforcement. An example of such a connection is illustrated in Figure 6-12.

Rather than extending the entire column reinforcing cage into the top of the drilled shaft, another approach which may be used to improve constructability is to use a shorter splice cage, sometimes called a 'jumper cage,' to provide a non-contact splice into the top of the drilled shaft and a lap splice into the column. This type of connection can also be advantageous where the column reinforcement is square or rectangular, as illustrated in the photo of Figure 6-14.

When the drilled shaft reinforcement includes a connection to a cap, grade beam, or abutment wall, it is important that the cage for the shaft should not include out-hook bars or other obstacles if temporary casing is used. In some cases, these can be turned inward during installation and then rotated into position after concrete placement is complete and temporary casing has been removed. Longitudinal bars can also be field bent hydraulically after the casing is removed. L-shaped bars or out-hooks could also be included into a secondary splice cage as described previously. The arrangement and spacing of the longitudinal bars of the drilled shaft must also accommodate passage of the bottom layer of rebars in the pile cap.

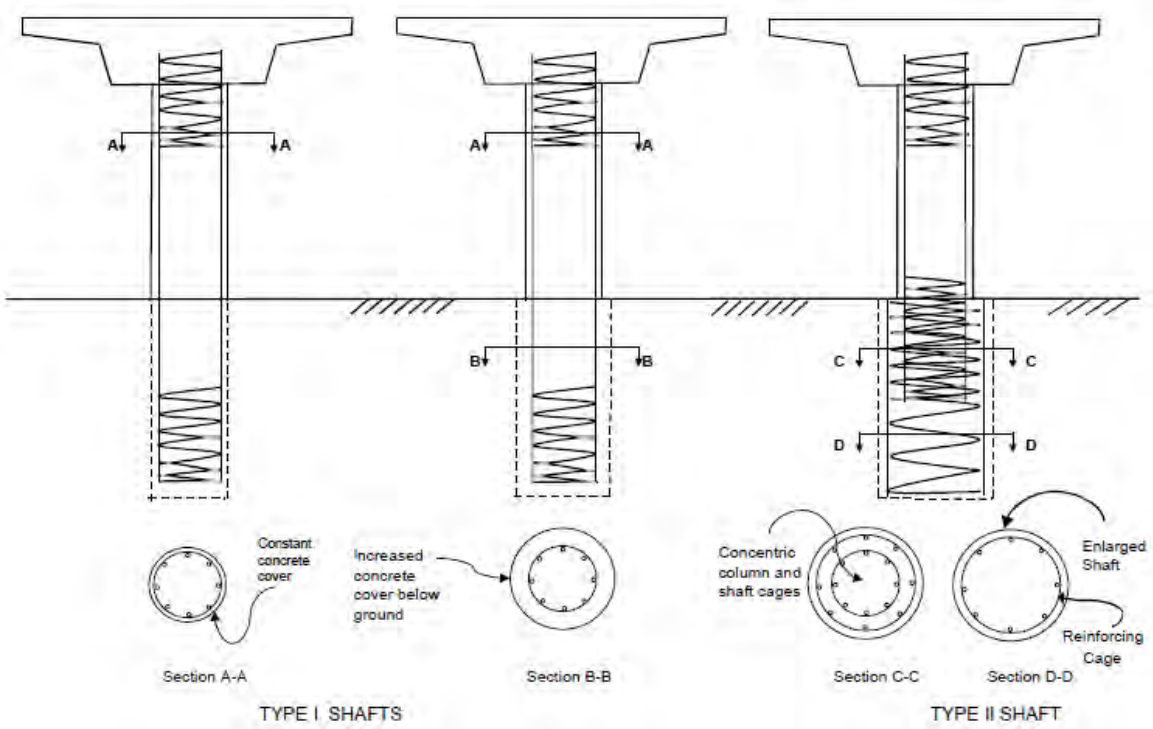


Figure 6-11 Type I and II Connections (from Caltrans Seismic Design Criteria, 2013)

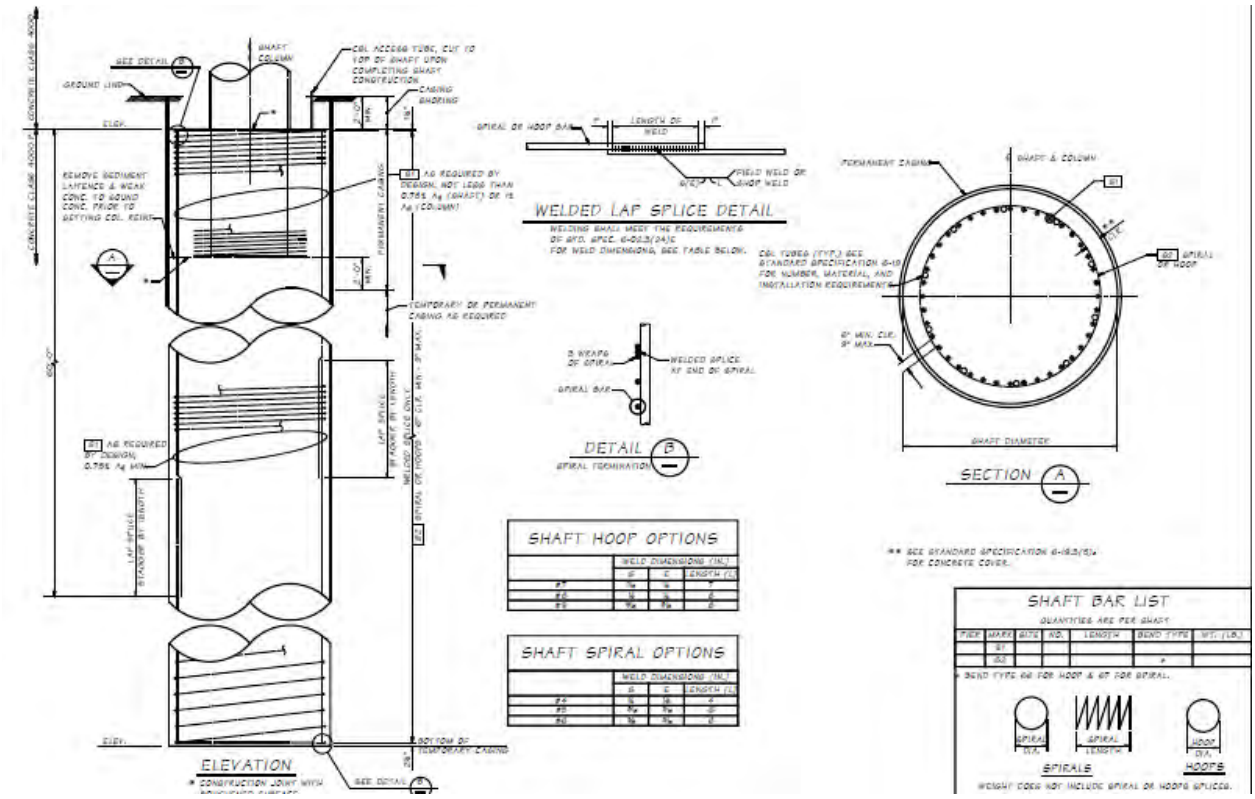


Figure 6-12 Washington DOT Type II Connection Detail (from WashDOT Bridge Design Manual, 2018)

A constructability issue can arise when a Type II connection must be fabricated using a single concrete placement in a wet hole environment, because the concrete would be required to flow through two cages. Even though appropriate openings are maintained in each cage, the openings will never line up from one cage to the next, and the opportunities for entrapping drilling fluids or poor quality concrete are significant. The best solution for construction of a Type II connection in a wet hole is to provide a short piece of permanent casing extending to a depth below the column reinforcement in order to allow a construction joint at the base of this splice as shown in Figure 6-13. When working over water, a short permanent casing combined with a larger diameter temporary casing or cofferdam can be used as illustrated in Figure 6-13. It is important that the shoring be of sufficient diameter to provide space for workers around the column formwork.

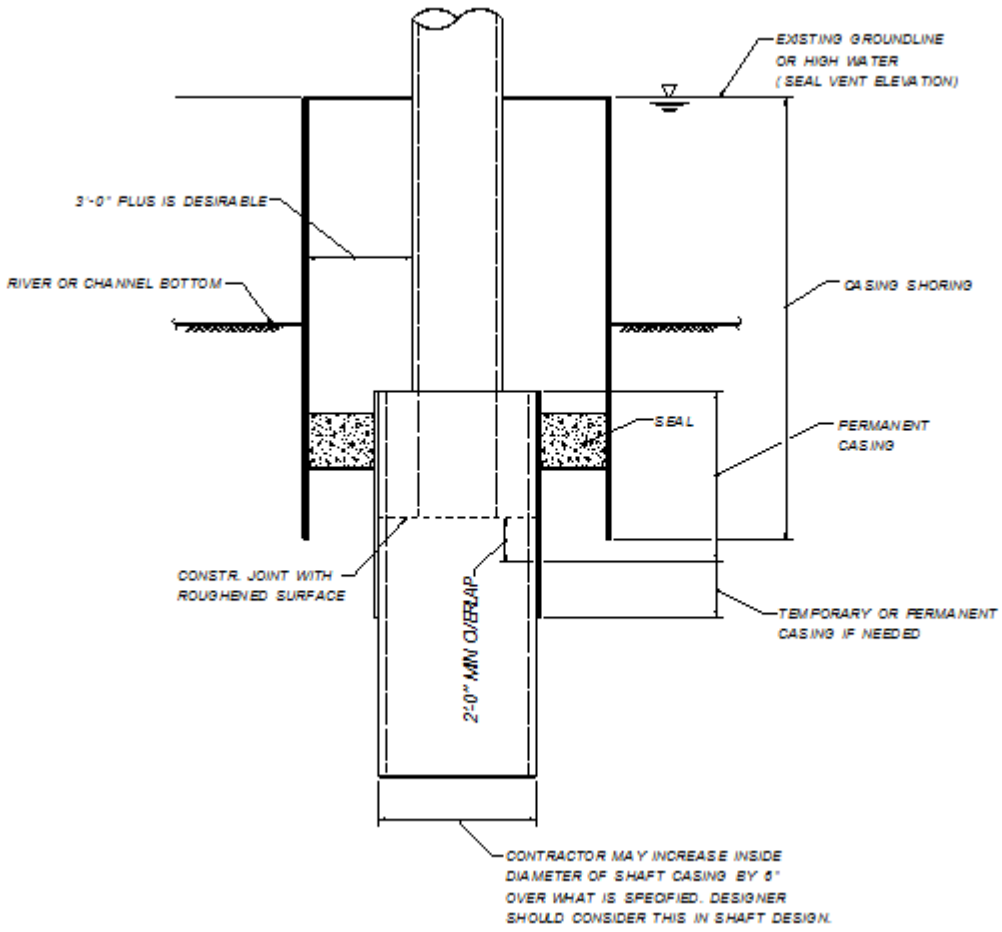


Figure 6-13 Construction of a Type II Connection Detail over Water (after ADSC West Coast Chapter)



Figure 6-14 Splice Cage Used to Fabricate Column to Shaft Connection

6.7 FABRICATION AND STORAGE

Drilled shaft rebar cages can be assembled at a fabrication plant or assembled on-site. Each has advantages and disadvantages. A fabrication plant provides a controlled environment and the potential for utilizing machinery to enhance efficiency and reliability. For example, welding can be performed faster and more efficiently in a manufacturing plant than in the field, using computer-controlled equipment. Precise control of dimensional tolerances is better achieved in a plant environment. The main disadvantages are associated with transporting the assembled cage to the job site. An extra cage-lifting operation is required to load the cage onto a truck at the plant. There are risks with transporting over-length loads and sometimes there are restrictions on such loads being transported on local roads. Nevertheless, there are situations where the benefits outweigh the cost and risk of transportation, and fabrication at a plant or other off-site facility is the best option. Figure 6-15 shows a crane off-loading a large cage which had been transported to a congested construction site alongside an existing freeway.

For most projects, the usual procedure is to transport the rebar to the job site and to assemble the cage reasonably close to where it will be installed. Cage transportation is eliminated, and handling of the completed cage is reduced to a minimum -- usually only to pick up the cage with a crane or cranes, and placing it in the shaft excavation. The photographs in Figure 6-1 through Figure 6-3 show workers fabricating rebar cages at job sites. The frames, or "jigs," that are shown for the temporary support of the cage are often necessary to fabricate large diameter cages correctly.

On rare occasions, the constructor may fabricate the cage directly over or in the drilled shaft excavation. The photos in Figure 6-16 show the fabrication of a large diameter cage by suspending the longitudinal bars from a "wind-chime" hangar frame and the adding the transverse hoops as the assembly of longitudinal bars is lowered into the hole. The example shown had permanent casing extending to rock. This procedure, however, should generally be avoided in uncased holes since it increases the time that the shaft excavation is open and increases the associated risks of hole instability and surface degradation.



Figure 6-15 Rebar Cage Being Delivered to Site



Figure 6-16 Photograph of Rebar Cage Being Assembled Over the Shaft Excavation

It is common practice to construct multiple cages prior to drilling the boreholes, and store them on-site until a particular cage is needed. This procedure allows the cage to be placed in the borehole in a timely manner following completion of excavation and inspection. Proper arrangements should be made to keep the stored cages free from contamination with mud or other deleterious materials.

Tying of the rebar is a critical aspect of cage fabrication. The purpose of tying the reinforcing bars is to maintain their position during handling of the cage and during concrete placement. Compared to many other reinforced concrete structures, drilled shaft construction imposes significantly more severe handling stresses on the rebar cage. During the lifting operation, stability of the cage is essential (first and foremost for safety) and proper tying is one of the factors that contributes to cage stability. In addition to the types

and frequency of ties specified in the rebar cage detailing, the effectiveness of tying also depends on the skill and experience of the workers assembling the cage and on proper quality assurance by qualified inspectors.

Tie wire connection types and materials are described in the publication *Placing Reinforcing Bars*, published by the Concrete Reinforcing Steel Institute (CRSI, 2016). Most tie wire is low carbon soft black annealed steel, which provides excellent flexibility needed for tying. Tie wire is available in gauges from 8 to 22; however, for rebar tying the most common gauges are 14 to 16.5. Considering the handling demands on drilled shaft rebar cages, No. 15 gauge (dia = 0.072 in) should be considered the minimum, and No. 14 gauge (dia = 0.083 inches) is recommended for larger bars in bundles and large-diameter cages.

CRSI (2016) identifies five basic types of ties at reinforcing bar intersections. The two tie types that provide the greatest holding strength are (1) wrap and saddle tie, and (2) figure eight tie with double wrap. These ties are recommended for heavy mats and cages that will be lifted by a crane, and are therefore applicable to drilled shaft rebar cages. Figure 6-17 shows an example of bundled No. 6 hoops tied to bundled No. 18 longitudinal bars using a figure eight tie.

For reinforcing configurations that will be lifted, for example preassembled mats, CRSI (2016) recommends that every bar intersection around the perimeter be tied and that alternate intersections be tied within the interior. Applied to drilled shaft rebar cages, this recommendation would require every intersection to be tied (since all are on the perimeter of the cage). Tying every intersection is the best practice for assuring cage stability and rigidity. However, research sponsored by Caltrans involving loading to failure of full-size cages described by Builes-Mejia (2010) found that internal bracing is the most significant factor affecting cage strength and stability, and that with properly designed internal bracing, tying frequencies of less than 100 percent were still effective. Internal bracing is discussed later in this chapter.

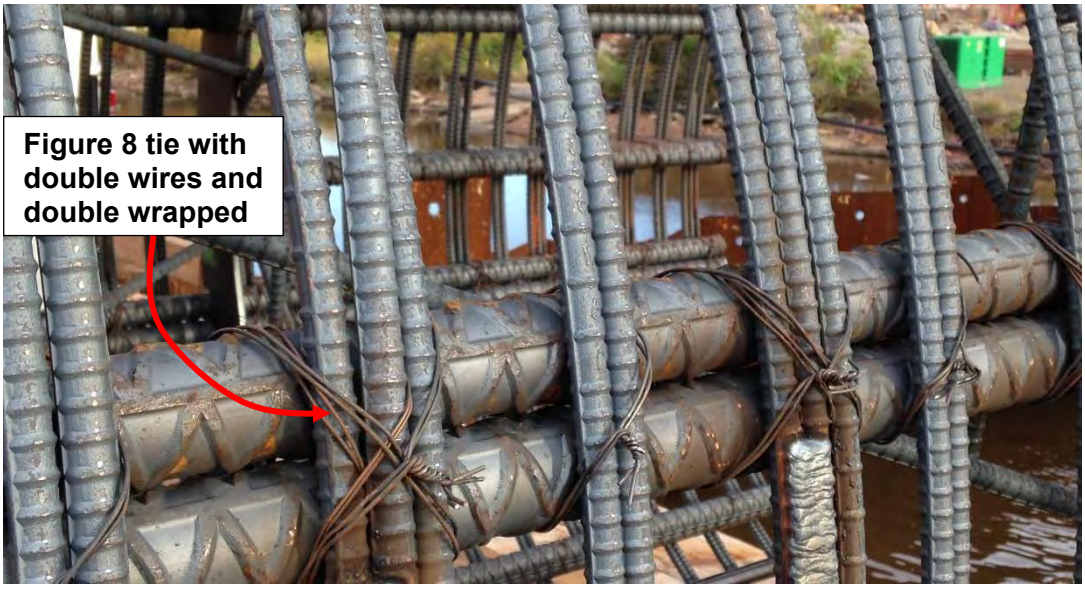


Figure 6-17Example of tie wire types to secure transverse reinforcing to longitudinal bars

Welding of rebar cages has not been widely practiced in the U.S., but is becoming more common, in particular for cages that are shop fabricated rather than assembled on-site. The cost of ASTM A706 steel, which is intended for welding, is marginally more expensive (about 5% more) than the more widely specified ASTM A615. Welded connections provide a generally stronger and more rigid cage.

Another factor affecting the ability of the cage to resist deformation during handling is the bar size of the spiral or hoops. ACI 318 (2014) specifies that the minimum bar size for transverse reinforcement is No. 3 for longitudinal bars smaller than No. 11, and minimum No. 4 transverse bars for longitudinal bars of No. 11 or larger. These minimum bar sizes may be sufficient for relatively small drilled shaft rebar cages; however, for purposes of stability during cage handling and concrete placement, larger transverse bars are often needed. Once the cage diameter reaches 4 ft, a minimum bar size of No. 5 is recommended for transverse steel. Hoops and spirals up to No. 7 and No. 8 are recommended for cages of 8 ft diameter and larger. In most locations, spiral is available in sizes up to No. 7, and hoops are available up to No. 8. Using larger bar sizes also allows larger clear spacing between the transverse steel and therefore promotes good constructability

Note also that distortion of the cage can occur as hydraulic forces pull the top of the cage downward and laterally if concrete flows to one side of the excavation to fill a void or oversized excavation. These cavities can be hidden by casing and then cause distortion of the cage during removal of temporary casing. Where the potential for these conditions exists (for example, in karstic limestone or rock where large overbreak is possible), then it is especially important that the cage be tied carefully, supported vertically, and properly braced during concrete placement and removal of casing. Bracing, stiffeners, and appropriate vertical support are described in Sections 6.10 and 6.11. The cage and concrete mix properties should also be designed to ensure that good passing ability is achieved.

6.8 SIZING HOOPS

Sizing hoops of the proper diameter are often used to aid in the fabrication of the rebar cage and to ensure that the finished cage diameter is correct. These hoops, also referred to as ‘gauge hoops’ or ‘template hoops,’ simply provide guides for the fabrication of the cage and can be made of plain rebar or thin rolled-plate stock. Sizing hoops are typically spaced at 8 to 10 ft longitudinally and tied at every intersection with the longitudinal bars. Sizing hoops can be made with a lapped splice as illustrated on the left side of Figure 6-18, or the ends of the hoop can be butt-welded, as illustrated on the right side of that figure. Marks on the sizing hoops are used to facilitate the placing of the longitudinal steel. Although sizing hoops give the finished cage some additional dimensional stability, they are not considered in the structural design and therefore butt welding on non-weldable steel is acceptable.

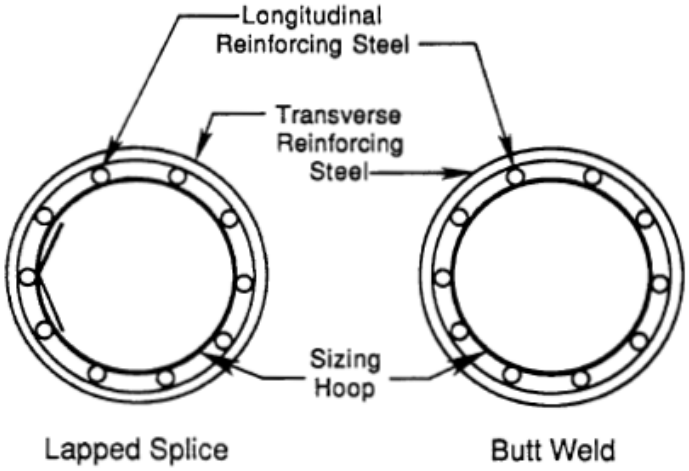


Figure 6-18 Sizing Hoop Assembly (from Le Labratoric Central des Ponts et Chaussees, 1986)

6.9 CENTERING DEVICES

The completed rebar cage must be sized to provide ample room for the fresh concrete to flow up the annular space between the cage and the sides of the excavation or casing, as well as to provide adequate concrete cover for the rebar. The most effective means to provide the required clearance and concrete cover is to use centering devices attached to the outside of the cage to keep the rebar at an appropriate distance from the walls of the borehole or casing. Centering devices may also be used on the interior of the cage to guide the tremie in a wet-hole concrete placement operation.

In accordance with AASHTO, minimum concrete cover should be 3 inches for drilled shafts with diameters up to 3 ft; 4 inches for diameters greater than 3 ft and less than 5 ft; and 6 inches for drilled shaft diameters of 5 ft and larger. In addition, the minimum annular space should be not less than five times the largest size of coarse aggregate in the concrete mix. As indicated in previous sections, a planned cover of 6 inches, or more, has advantages for individual shafts supporting a single column. The centering device selected for a given shaft must account for the intended concrete cover.

A variety of devices have been used for centering drilled shafts. Centering devices that can rotate (rollers) as the cage is lowered into the excavated hole are most effective in minimizing the risk of dislodging soil or debris which can then accumulate at the bottom of the excavation prior to concrete placement. This is an important consideration since base cleaning becomes difficult or impossible once the cage is installed. Disturbance of the sidewall could also affect side resistance adversely. Properly sized and secured roller-type spacers are also less likely to be dislodged during cage installation. Roller spacers are typically constructed of plastic, concrete, or mortar; they should not be fabricated of steel in such a way that a corrosion path to the reinforcement could be introduced. Examples of centering rollers are provided in the photos of Figure 6-19. Rollers are typically available in diameters that provide 3, 4 or 6 inches of spacing on the outside of the cage.



Figure 6-19 Roller Centralizers on Reinforcing Cage. Clockwise from top: cage with external roller centralizer being installed in a cased hole; evenly-spaced rollers attached to hoop reinforcement; extracted shaft revealing centralizer on surface.

Flat or crescent shaped centralizers ("sleds") are appropriate for sections of the cage that will be installed in casing, but should not be used in uncased portions of shafts since they increase the risk of material being dislodged from the side of the excavation and accumulating debris at the base of the drilled shaft excavation. Figure 6-20 shows examples of sled centralizers. The photograph in Figure 6-20 (b) shows concrete sleds maintaining the required spacing between the cage and casing.



(a) Plastic Sled (b) View Inside Casing Showing

Figure 6-20 Sled-Type Centralizers.

Some specifications also call for the base of the drilled shaft cage to be prevented from contacting the soil or rock at the bottom of the borehole for corrosion protection. Small concrete, mortar or plastic 'feet' or 'chairs' can be made or used for this purpose, as illustrated in Figure 6-21. The photo at right shows the base of a concrete shaft after the shaft was extracted (for research purposes) in which the plastic chairs are evident. These devices can also be used to reduce bearing pressure under the longitudinal bars from the weight of the cage and prevent the rebar from penetrating into the soil in a case where the weight of the cage is supported on the base of the excavation.



Figure 6-21 Chairs or "Feet" for Base of Reinforcing Cage

6.10 BRACING OF REBAR CAGES

A critical stage in the construction of a drilled shaft occurs when the cage is lifted from a horizontal position on the ground (its orientation when fabricated or delivered), rotated to vertical, and lowered into the

borehole (referred to as 'tripping' the cage). Bracing is used to increase the strength and stiffness of the cage during lifting operations. Both internal and external bracing systems have been used successfully. As drilled shafts used in transportation applications have increased in length and diameter, with corresponding increases in rebar cage sizes, internal bracing systems have become the preferred method for stiffening the cage. Research and experience have led to improved guidelines for selecting appropriate bracing.

A basic rule-of-thumb is that internal bracing is required for rebar cages with a length to diameter ratio of 8 or greater. Two configurations of internal bracing are used: X-Type bracing or Box/Square bracing. X-Type bracing may interfere with concrete placement and may therefore need to be removed as the cage is being lowered into the excavation, making the installation more time-consuming. Box or Square bracing can be configured to provide an opening sufficient for concrete placement by tremie or free-fall, allowing the bracing to be left in place permanently. Figure 6-22 shows a cage with square internal bracing. This cage also has internal stiffening rings spaced at 10-ft longitudinally, which provides a means to attach the bars forming the square bracing, as well as providing additional strength and rigidity to the cage. At each ring location, two boxes, offset from each other by 45 degrees, are attached to the ring in this case by welding, but could also be tied to the longitudinal bars. In addition to the bars forming the box, which are oriented perpendicular to the longitudinal bars, diagonal bars connect the rings longitudinally around the inside perimeter. As can be observed in the photo, the opening inside the cage provides ample space for the tremie pipe. A minimum bar size of No. 8 is recommended for this type of internal bracing. Note that the rings can double as template hoops for fabricating the cage.



Figure 6-22 Rebar Cage with Square-Type Internal Bracing and Stiffener Rings

6.11 REBAR CAGE LIFTING AND PLACEMENT

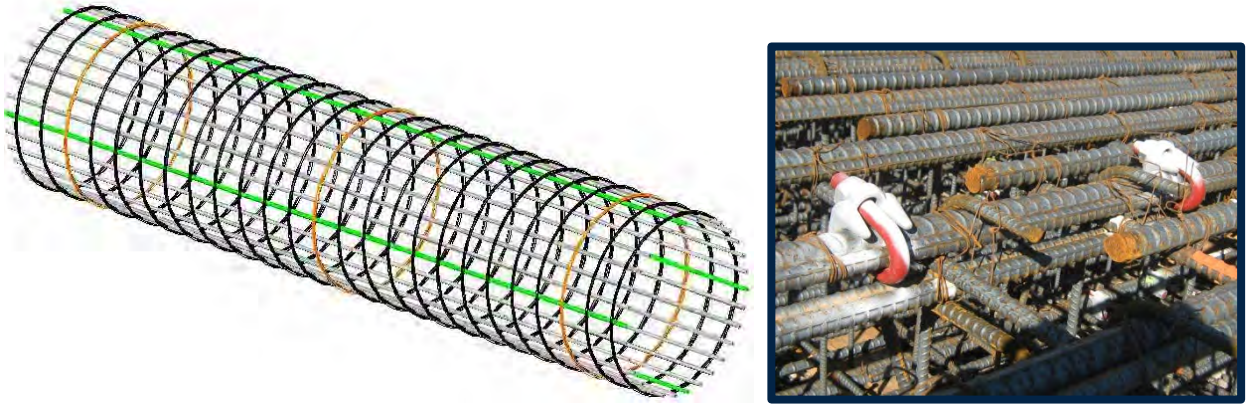
Lifting the cage and tripping it to the drilled shaft location for placement in the hole is a critical step in the installation process. Safety is the paramount consideration, but it is also worth noting that any incidents involving cage instability or accidents also have negative impacts to the construction schedule, and may

also lead to regulatory agency investigations and/or fines. It is therefore in the best interest of all parties to make sure that cage installation is a well-engineered, well-executed, non-eventful operation.

Some of the features of a cage that can facilitate safe handling include pickup bars and lifting collars or frames. Pickup bars are separate, additional longitudinal bars, typically 4 each that form a square (every 90 degrees circumferentially) and which are dedicated bars for rigging point attachments (Figure 6-23a). Pickup bars should be tied at every intersection using a double or quadruple snap tie or a double saddle tie. Alternatively, the pickup bars can be fastened to the other longitudinal bars using Crosby Clips or a similar mechanical connector (Figure 6-23b).

Collars are used at the top of a cage to provide a strong attachment between the crane rigging and the cage. During lifting, the collar is one of the attachment points and shares the load of the cage with the other attachment points. Once the cage is vertical, its entire weight is supported through the collar making this a critical connection. Examples of lifting collar configurations are shown in Figure 6-24.

Prior to lifting, it is the contractor's responsibility, with engineering support, to select the appropriate crane(s), establish the weight and center of gravity of the cage, and select the appropriate number and locations of rigging points. The rigging point connections should be designed for a factor of safety of 2 or greater.



(a) (b)

Figure 6-23 (a) Configuration Showing Layout of Pickup Bars (shown in green); (b) Crosby Clip Connection of Pickup Bar to Longitudinal Bar (Graphic and Photo courtesy of V. Siebert, Siebert and Associates, LLC)



(a) Lifting Ring (b) Lifting Frame

Figure 6-24 Examples of Lifting Collars

Figure 6-25 shows a well-executed lifting sequence from horizontal to vertical position over the drilled shaft excavation. A single crane lift is being used, with two blocks, each with a spreader bar, one with 4-point rigging and the other with two-point rigging. Note that the attachment points on the cage correspond to the locations of stiffener rings. Although some small amount of distortion is visible as the cage is being rotated, the magnitude of deformation is in the elastic range, and the cage is free of any permanent distortion when it reaches the vertical position. Note also that the entire cage is lifted off the ground so that the bottom of the cage is not being dragged, which is a potential cause of damage that should be avoided.

Figure 6-26 shows an example of a well-executed cage lift, in this case using two cranes with five rigging points. For very large cages an effective means to lift and place the cage is a device commonly referred to as a 'tipping frame' or 'lift table,' which supports the cage and rotates it upward. Figure 6-27 illustrates the use of a tilt table mounted on a barge to help lift the cage into the vertical position in a marine application.

Guidelines for the type of rigging and equipment needed for various cage lengths are summarized in Table 6-3, based on Siefert et al. (2016).

Table 6-3 GUIDELINES FOR LIFTING REQUIREMENTS (Siefert, et al., 2016)

Cage Length	Rigging/ Equipment Requirements
< 20 ft	1 block, 1 point rigging
20 to 40 ft	1 block, 2 point rigging
40 to 140 ft	2 blocks or 2 cranes, multi-point rigging
> 140 ft	Lift Table is most appropriate

Elastic deformation of a cage during lifting, such as shown in Figure 6-25 and Figure 6-26, is of no great concern; however, if plastic (permanent) deformation occurs or slippage of the ties or spiral is evident after the cage is brought to the vertical, the cage must be repaired before placing it in the borehole.

Once the cage is placed and centered properly in the excavated borehole, it can be supported vertically by several means. If permanent casing is being used the weight of the cage can be transferred to the casing by placing horizontal members between the top of the cage and top of casing. Figure 6-28 shows a cage being supported in this manner. In this case the lifting ring at the top of the cage serves as a means to transfer the cage weight to the top of the casing. When the construction operation requires that the cage be self-supported by standing the cage on the bottom of the shaft excavation, it is particularly important that the cage be well-tied and free of distortion from the lifting operation.

Following lifting of the rebar cage, additional roller centralizers should be attached to the rebar cage to replace those damaged or missing from the cage.



Figure 6-25 Example of Proper Cage Lifting Sequence, Single Crane Setup



Figure 6-26 Rebar Cage Being Lifted Properly Using Two Cranes



Figure 6-27 Photograph of Rebar Cage Being Lifted with a Tipping Frame (photo courtesy Malcolm Drilling)



Figure 6-28 Rebar Cage Being Supported by Permanent Casing

6.12 SUMMARY

This chapter provides an overview of the factors to be taken into account to design rebar cages for constructability. Properties of reinforcing steel used in drilled shafts are reviewed. Features of drilled shaft cages, including longitudinal reinforcing, transverse reinforcing, splices, and connections to the pier column or foundation cap are described, all with a focus on practices and special considerations that promote constructability while satisfying design requirements. Compared to columns or other reinforced concrete structures, drilled shaft construction presents special considerations for handling and fabrication of the cage. The design of the cage must ensure concrete passing ability and provide for construction tolerances. The design of splices and connections to the structure must include considerations for construction procedures. The handling and placement of the reinforcement into the drilled shaft excavation must be planned and executed with care to ensure that the structural requirements provided by the reinforcing are achieved. These many and sometimes conflicting considerations can only be addressed effectively if engineers have a good understanding of the special construction requirements of drilled shafts, and if constructors are properly equipped and trained, and have a well-designed installation plan.

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CHAPTER 7
PLACEMENT AND DESIGN OF CONCRETE FOR DRILLED SHAFTS

7.1 INTRODUCTION

Construction of a drilled shaft foundation can be thought of as fabrication of a reinforced concrete element in-situ. This aspect of the manufacture of the foundation structure is often conducted under extremely challenging conditions with placement of reinforcing steel and concrete at depths exceeding 100 ft below the ground surface, often in a borehole filled with water or drilling slurry. Concrete placement techniques and materials represent a critical aspect of the process and require thorough planning and design to assure the geotechnical and structural performance of the completed shaft. While mix design, and the means and method of concrete placement are most often delegated to the contractor, it is imperative that all parties in the process have a good understanding of the basic requirements and characteristics of drilled shaft concrete.

This chapter describes placement of concrete for drilled shafts, and the design of concrete mixes with emphasis on the unique requirements for drilled shaft concrete. This chapter focuses on specific issues of greatest import to drilled shaft construction and performance, emphasizing the workability characteristics critical to success in this application. Testing for quality control and quality assurance during batching and concrete placement is described in this chapter; inspection of concrete placement is described in Chapter 15 while integrity testing of completed drilled shafts is described in Chapter 16.

Drilled shaft concrete and aspects of construction of drilled shafts related to concrete are discussed in several references that are recommended as parallel reading for this chapter. A recent publication by a joint task force of EFFC (European Federation of Foundation Contractors) and DFI (Deep Foundations Institute) provides an overview of the fundamental characteristics of fluid concrete, a state of the practice (2018) summary of good practices for design and construction with tremie concrete, and a summary of recent research into the flow characteristics of tremie-placed fluid concrete. The publication "Design and Control of Concrete Mixtures" (Kosmatka and Wilson, 2016) provides appropriate background for readers not familiar with concrete mixtures in general or with terminology related to concrete. ACI 336.1 (2001) provides a description of concrete for drilled shafts, although a bit dated relative to recent developments with concrete mix design and admixtures for fluid concrete.

7.2 BASIC REQUIREMENTS FOR DRILLED SHAFT CONCRETE

Because of the unique construction conditions and techniques used for drilled shafts compared to other reinforced concrete structures, the concrete used must be designed for the specific requirements of this application. The most important of these are the special workability requirements for the fresh concrete during transport and placement operations. Often, fresh concrete must be transported long distances to a remote bridge site to be pumped long distances, to then flow readily through a tremie and congested reinforcement under slurry to fill a hole that may be 10-ft diameter at depths exceeding 100 ft. Additionally, the mix may be required to remain workable for periods of 4 to 8 hours or more in widely ranging ambient temperature conditions. The fresh concrete must also consolidate under its own self-weight without vibration and without segregation, excessive bleeding, or excessive heat of hydration. Such requirements are challenging to be sure, but not uncommon for modern drilled shaft construction.

The basic requirements for drilled shaft concrete can be summarized as follows:

- Workability: The concrete must have the ability to flow readily and fill the shaft excavation completely. The concrete must readily pass through the reinforcement without blocking to achieve thorough contact with the surrounding soil or rock. The concrete must be self-leveling within the excavation and consolidate under self-weight; vibration of concrete in a borehole is not possible or practical.
- Workability Retention: With underwater tremie placement, or when casing must be withdrawn after completion of concrete placement, the drilled shaft concrete must retain workability and have controlled setting times suitable for completion of placement operations.
- Stability: While providing the high degree of workability required, the fresh concrete must still have robust stability and resist any tendency to segregate and bleed. The paste within the concrete mix should have a high degree of cohesion so that coarse aggregate particles are evenly distributed through the mix, and water within the mix should remain distributed without a tendency to bleed and result in non-uniform properties or bleed water channels.
- Durability: The concrete cover on the reinforcement must provide low permeability so as to minimize the potential for corrosion of the reinforcement. If the subsurface environment is aggressive or may become aggressive during the life of the foundation, the concrete should be designed to have high density and low permeability so that the concrete is able to resist the negative effects of the environment.
- Appropriate Strength and Stiffness: The concrete must provide the strength and stiffness necessary to meet structural performance requirements.

In most cases, drilled shafts are not subject to large structural stresses within the concrete so strength demands are relatively ordinary compared to the extreme workability requirements. The following section therefore highlights concrete placement issues and outlines recommended construction practices. A complete understanding of concrete placement requirements during construction of drilled shafts is necessary to develop a mix design with appropriate workability characteristics for the specific needs of a project. A discussion of concrete mix design therefore follows the section addressing concrete placement.

7.3 PLACEMENT OF CONCRETE

Concrete placement in a drilled shaft excavation must be carefully planned and executed to meet the specific conditions associated with the different methods of construction described in Chapter 3. The following sections describe procedures used for concrete placement in dry and wet excavations, and with the considerations necessary for temporary casing.

7.3.1 Placement in a Dry Shaft Excavation

A dry excavation without temporary casing provides the simplest conditions for concrete placement. In general, concrete may be placed by free fall methods as long as the concrete is directed down through the center of the shaft without directly hitting the reinforcing cage or the sides of the excavation as illustrated in Figure 7-1. Impact of the fluid concrete against the sides could potentially produce distortion of the reinforcing cage or dislodge soil or debris into the fresh concrete during placement.

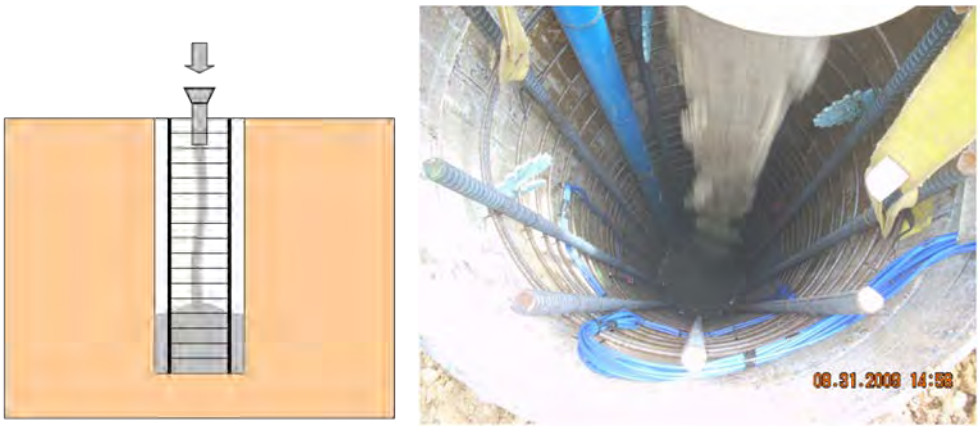


Figure 7-1 Free Fall Concrete Placement to Avoid Concrete Impacting Reinforcement.

The flow of concrete in free fall should be directed to the center of the borehole and cage by a drop chute or other acceptable device to keep the stream of falling concrete centered in the hole. Laborers with shovels are not generally able to direct the stream of concrete adequately. Similarly, use of a flexible hose is not recommended because of the difficulty in directing the discharge from the hose. A drop chute composed of a short section of relatively stiff pipe can be used to effectively direct the flow of concrete into the center of the shaft excavation. In cases where the concrete ready-mix truck can be positioned near the top of the excavation, the last section of the chute from the truck can often be used to direct the flow as illustrated in Figure 7-2. Figure 7-3 illustrates a case in which the concrete was lifted to the top of the column reinforcement using buckets, and placed into the dry excavation via a drop chute. In this example, the drop chute is a simple stiff plastic tube attached to a hopper or funnel at the top. A tremie pipe (described in Section 7.3.3) may also be used as a simple drop chute for a dry shaft excavation.



Figure 7-2 Free Fall Concrete Placement Using Ready-Mix Truck Chute.

Discharge of concrete into the relatively small space comprising the drilled shaft excavation eliminates issues related to segregation from free fall placement onto a flat surface, and a number of detailed studies have conclusively shown that free fall placement does not cause segregation or other significant adverse effects on drilled shaft concrete. Kiefer and Baker (1994), Bru et al. (1991), and Baker and Gnaedinger (1960) report on field studies of free fall placement that show that concrete mixes designed for dry placement can be dropped freely for distances up to about 80 ft without problems. However, highly

flowable concrete typically used for tremie or pumping placement methods (i.e., "tremie concrete" as described in Section 7.5.4) should not be placed by free fall because it may segregate.



Figure 7-3 Placement into a Dry Excavation Using a Drop Chute

If concrete is placed by free fall into a shaft excavation that is not completely dry, there will be mixing of the concrete with the water present at the base of the shaft. The result will be a concrete mix with excessive water or perhaps even a zone of washed aggregate if a substantial amount of water is present. The contractor may be able to remove the water by pumping, and a small amount of water at the bottom of the shaft excavation (typically specified to be less than 3 inches deep) is acceptable provided there is no substantial inflow of water. In general, a flow into the excavation producing greater than 12 inches of water per hour (1 inch per 5 minutes) is considered excessive. If excessive seepage occurs, as shown in Figure 5-23, it is necessary to flood the excavation to stop the inflow, and to place concrete using the wet method as described in Section 7.3.3. It is not sufficient to simply use a tremie without flooding the excavation to control inflow, because water inflow will drip onto the rising surface of the concrete, contaminating the concrete and potentially causing defects in the completed shaft. Even when the surface of the concrete passes the location of inflow, a groundwater pressure head greater than the head of concrete during the placement operations could result in formation of flow channels of water into or through the fresh concrete. By flooding the excavation and placing concrete with a tremie, the higher fluid head within the shaft excavation will maintain a positive outward direction of flow until the fluid concrete has filled the hole.

7.3.2 Placement in a Dry Shaft Excavation within a Cased Hole

The previous section described placement of concrete within an open, dry shaft excavation. In some cases, the excavation may be dry, but the dry condition is achieved using a temporary casing that extends through water-bearing strata. Additional considerations are necessary for concrete placement in a cased hole in which temporary casing will be removed.

Where temporary casing is used to seal a dry shaft excavation, there may be an accumulation of fluid on the outside of the casing. When the casing seal is broken to remove the casing, the head of concrete inside the casing must be sufficient for the concrete pressure to exceed the fluid pressure on the outside of the casing, and this concrete head must be maintained to avoid the potential for inflow of fluids into the fresh concrete (i.e., a breach of the casing). The head of concrete within the casing will drop as concrete flows out to fill the annular space and any voids outside the casing as illustrated in Figure 7-4. Note that fluid outside the casing is often heavily laden with silt and sand such that the unit weight may be significantly greater than the unit weight of clean water. In addition, the concrete pressure at the base of the casing will be somewhat less than the measured head within the cage near the tremie due to losses in head across the reinforcing cage as the concrete flows out of the casing to fill the space outside the casing. Therefore, it is necessary to maintain a substantial margin of excess head above the theoretical computed head difference illustrated in Figure 7-4 to minimize risks of water or slurry inflow into the drilled shaft concrete.

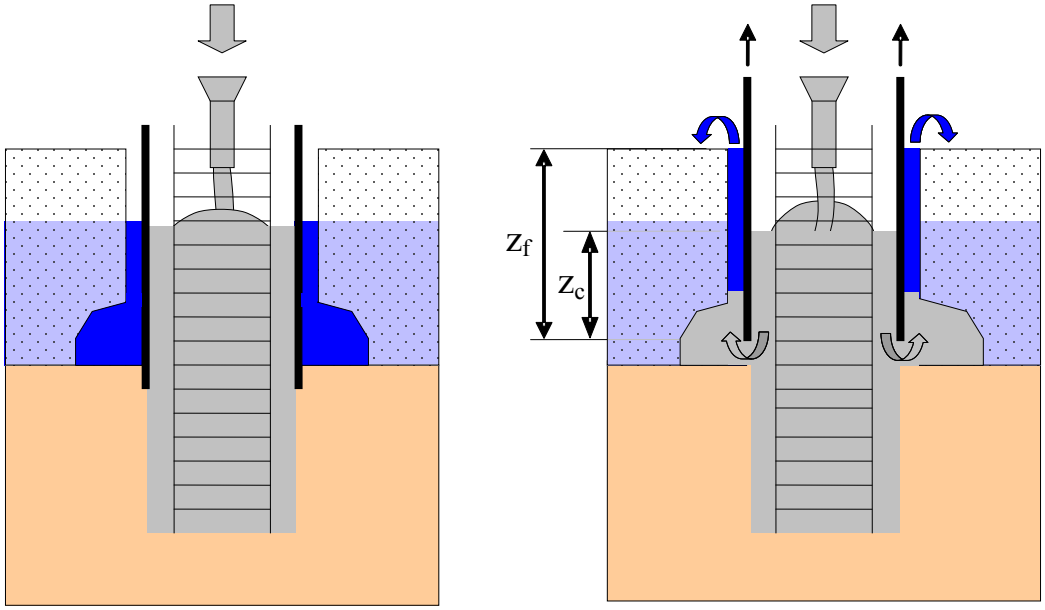


Figure 7-4 Concrete Pressure Head Requirement During Casing Extraction: (a) Prior to Lifting Casing; (b) as Casing is Lifted

The requirement to provide sufficient concrete to overcome external fluid pressure is complicated by the possibility that there could be overbreak or cavities of unknown size outside of the casing. When the casing is pulled and the seal into the underlying formation is eliminated, the head of concrete within the casing will drop immediately due to the volume required to fill this space. It is therefore essential that concrete be supplied at a sufficient rate to maintain a positive head of concrete inside the casing in excess of the fluid head to the ground surface outside the casing, as illustrated in Figure 7-4. The photo in Figure 5-11 illustrates the expulsion of fluids to the surface from the space around the casing as fluid concrete fills the excavation from the bottom.

It is also essential that the concrete have good workability throughout the duration of placement operations. The concrete must flow easily through the cage to displace fluids and completely fill the excavation. Even with dry conditions outside the casing, a loss of concrete workability prior to casing extraction can result in arching within the casing such that concrete is lifted and a "neck" occurs below the casing. This condition can also result in displacement of the reinforcement or inability to remove the

casing. Concrete that does not flow readily through the reinforcement will tend to load the cage vertically and may cause racking or distortion in the reinforcing cage. If there is a void outside the casing on one side of the shaft, concrete with poor workability may tend to drag the cage toward the side with the void, resulting in downward movement and distortion of the cage.

Where telescoping temporary casing is used, it is important that concrete fill the void space from the bottom up as each section is removed. The concrete should flow from the inside out, and casing extraction must be carefully managed to prevent entrapment of water within the concrete. An illustration of extracting a telescoping casing during concrete placement is provided in Figure 7-5. Note that the initial placement of concrete into the deeper casing (Figure 7-5a) must be directed to prevent spillage of concrete over the lower casing into the annular space between casings. The contractor may choose to first extract the inner casing as shown in Figure 7-5b, or may choose to remove the outer casing first while maintaining a head of concrete within the inner casing. In either case, the concrete head must be maintained within the casing sufficient to exceed the head of any fluid outside the casing. In either case, the removal of the outer casing introduces the opportunity for concrete to flow out and fill overbreak volume as described previously.

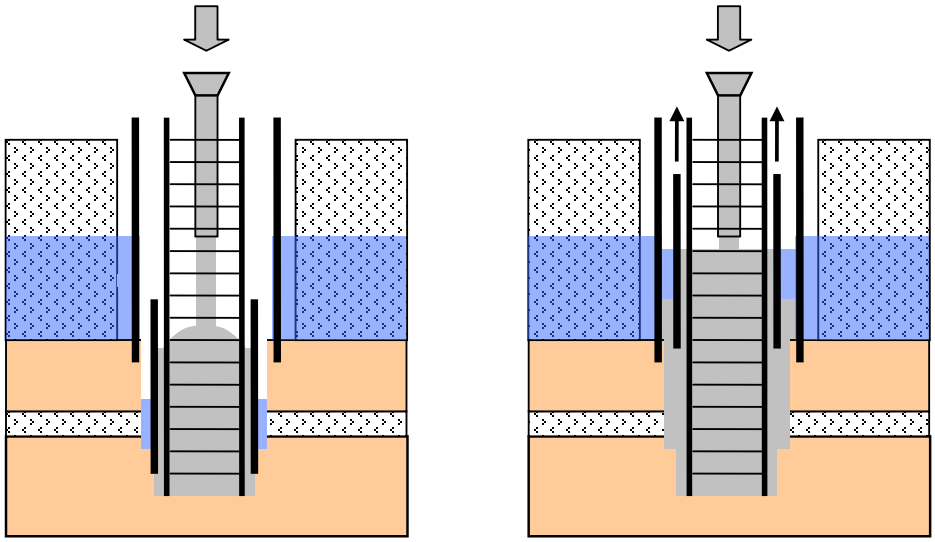


Figure 7-5 Concrete Placement with Telescoping Casing: (a) Initial Placement of Concrete; (b) Initial Extraction of Inner Casing

7.3.3 Placement of Concrete in a Wet Excavation

Where a wet excavation is required for construction, or a sufficiently dry excavation cannot be maintained as described in Section 7.3.1, concrete must be placed using underwater techniques with either a tremie or pumpline. Underwater placement of concrete generally requires use of concrete mixes that are different from those appropriate for dry placement; in this manual, concrete suitable for underwater placement is referred to as "tremie concrete." Regardless of whether the concrete is placed with a tremie or pumpline, it is essential that tremie concrete have good workability for the duration of placement operations and that the bottom of the concrete delivery tube be maintained sufficiently below the rising surface of fresh concrete. Both tremie and pumpline systems have been used successfully, although the gravity tremie is more common, especially with relatively deep (greater than 60 ft) shafts.

7.3.3.1 Placement by Gravity Tremie

A gravity-fed tremie is a steel tube, usually with a hopper on the top, that is fed from a pump, by discharging from a bucket, or directly from a ready mix truck. Aluminum should never be used because of reactions with the concrete, and plastic pipe such as PVC is generally not sufficiently robust. The diameter of a tremie tube for gravity placement of concrete depends on the diameter and depth of the excavation; tremie pipes with an inside diameter of 8 to 12 inches are most common, although larger diameters may be used. A 10-inch diameter tube is generally the smallest that should be used for a gravity tremie.

The tremie must be watertight to prevent inflow of drilling fluid during concrete placement, and must have a smooth and clean inner surface to minimize drag on the concrete flow. A tremie with obstructions or hardened concrete on the inside will increase frictional resistance to the concrete flow and may cause a blockage. The tremie in Figure 7-6 is contaminated with hardened concrete and must be cleaned prior to use, or discarded. A smooth outer surface is generally desirable to avoid entanglement with the reinforcing cage, although tremies with flanges protruding on the outside (such as those in Figure 7-6) have been used successfully if the verticality of the tremie can be controlled and the cage is large enough to permit passage of the flanges.



(a) (b)

Figure 7-6 Tremie segment: (a) Contaminated with Hardened Concrete; (b) Worker Cleaning Tremie

In a relatively short shaft (usually less than 40 to 50 ft long), a solid, one-piece steel tube may be used as a tremie as shown in Figure 7-7. Deeper drilled shafts typically require use of a sectional or segmental tremie. A segmental tremie is assembled from sections with waterproof joints as illustrated in Figure 7-8. Several types of joints are available, usually designed to include an o-ring seal. Segmental tremies can be disassembled as they are being extracted from the excavation, which minimizes the height that concrete must be pumped or lifted by bucket to charge the tremie.

With water or slurry in the excavation, the concrete flow from the tremie must be initiated so that contamination of the concrete is minimized. Two general procedures may be used:

1. A closed tremie may be installed with the bottom of the tremie sealed with a cover plate
2. An open tremie may be installed and a traveling plug inserted ahead of the concrete.

Both methods require careful attention to details during initiation of concrete placement in the shaft.



Figure 7-7 Solid Tremie Pipes



Figure 7-8 Assembly of Segmental Tremie for Concrete Placement

The closed tremie is placed into the shaft, concrete placed within the tremie, and then the tremie opened to release concrete. The closed tremie must be watertight to avoid mixing of concrete with water or slurry inside the tremie. After placement of the tremie into the shaft, the inside of the tremie should be visually checked for leaks before placement of concrete into the tremie. The buoyancy of a watertight closed tremie is one limitation of the use of this method relative to an open tremie; in some situations, it may be necessary to add weight to make the closed tremie sufficiently heavy to overcome buoyancy.

A seal at the bottom of a closed tremie is normally provided using a sacrificial closure plate. This closure plate can be a simple steel plate, sometimes with a rubber gasket to help seal the closure. The closure plate can be duct-taped onto the flat, smooth base of the tremie pipe as shown in Figure 7-9, or covered with a plastic wrap and tied to the tremie. Since the fluid pressure acts against the closure plate from the outside, the tape does not require great strength to hold the plate on the tremie; in fact, excessive duct-tape can make it difficult to break the plate off during concrete placement. With a fluid pressure acting on the plate from the outside approaching 1/2 psi per foot of depth in the hole, the plate must have sufficient strength to resist the hydrostatic head of the drilling fluid. Other types of closure plates have been used,

including a pan-type or “hat” device, which fits like a cup over the outside of the tremie with an o-ring seal. Some contractors have even used a plate with a hinge system so that the closure plate remains attached to the tremie, although a projecting hinge has the potential to hang on the rebar cage as the tremie is lifted.



Figure 7-9 Closure Plate for Closed Tremie

Concrete placement with a closed tremie is started by first placing the tremie to rest on the bottom of the shaft. The tremie is then filled with concrete and lifted to break away the closure plate, which produces a surge of concrete when the tremie is first pulled upwards a small distance (about 6 to 12 inches). This "rush" of concrete occurs because the pressure due to the weight of concrete within the tremie is much greater than the fluid pressure outside of the base of the tremie. The inertia of the concrete forces its way under the fluid at the base of the excavation and pushes the drilling fluid out at the top.

It is important that the closure plate release freely when lifting the tremie. There have been reported instances of difficulties in releasing the closure plate when the plate is heavily duct-taped to the bottom of the tremie. If the tremie is lifted more than a few inches above the base, there is a risk of the concrete becoming contaminated as a result of falling through the drilling fluid.

With an open tremie, the open pipe is installed into the drilling fluid and held a few inches from the bottom of the shaft excavation. Prior to introduction of concrete, a traveling plug commonly called a “pig” or “rabbit” is placed into the tremie pipe to act as a separator between the drilling fluid and fluid concrete to prevent mixing as the concrete travels down the tremie pipe. The plug may be constructed of polystyrene, closed cell foam, or foam rubber that has been saturated with water. The plug should not be so compressible that it fails to provide separation within the tremie pipe under the anticipated hydrostatic pressure. However, an excessively long plug will require that the tremie be lifted a considerable distance off the bottom for the plug to clear the tremie, and this lifting would allow concrete to fall through drilling fluid resulting in contaminated concrete.

Management of the tremie during the first few feet of concrete placement is a particularly important aspect of successful drilled shaft construction. It is vital that concrete delivery be continuous during this period until a head of at least 10 ft of concrete above the tip of the tremie is achieved. The tremie must be kept within a few inches of the bottom of the shaft during this period so the flow of concrete out of the tremie is controlled and a head of concrete inside the tremie is developed and maintained, as illustrated in

Figure 7-10. This control is especially important in deep, large diameter shafts, where a large volume of concrete is required to fill the tremie and the shaft excavation. If a limited initial charge of concrete is supplied and allowed to flow freely from the tremie, it is possible that the head of concrete within the tremie may not be maintained and a "back-surge" of drilling fluid can enter the tremie thus resulting in a breach of the tremie seal into concrete. Subsequent delivery of concrete into the tremie would result in mixing of concrete with drilling fluid and contamination of a substantial volume of concrete within the hole, as illustrated in Figure 7-11. It is therefore good practice to require that sufficient concrete be on site for developing the initial head of concrete within the shaft prior to commencing concrete placement.

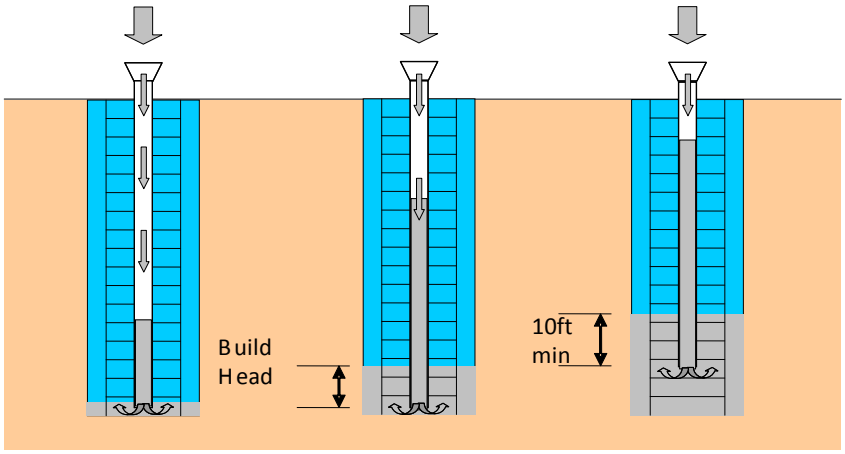


Figure 7-10 Control of Tremie to Establish Concrete Head

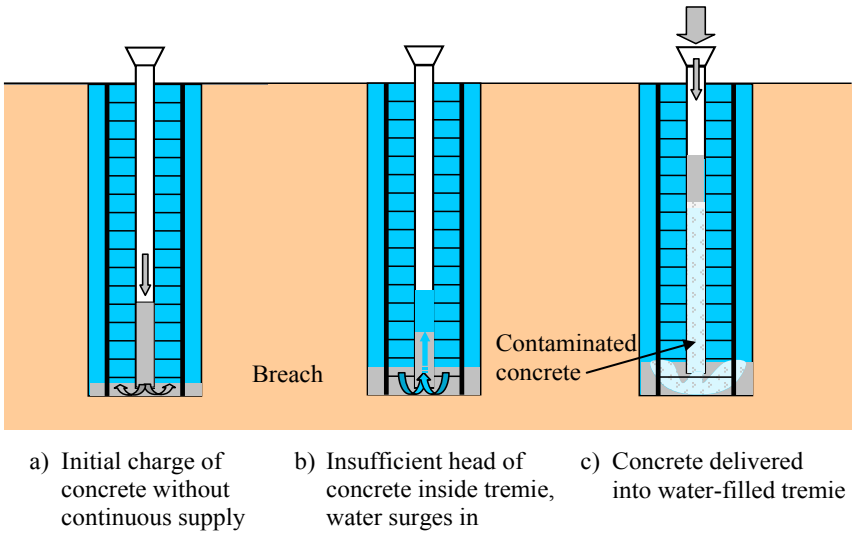


Figure 7-11 Breach of Tremie Due to Failure to Establish Concrete Head

During tremie placement, the end of the tremie should remain embedded a minimum of 10 feet into the fresh concrete throughout placement. As the column of concrete rises within the shaft, the tremie should be lifted as required to maintain flow. The contractor will need to use a segmental tremie or provide the capability to lift concrete to the top of an elevated solid tremie in order to lift the tremie during concrete placement. If the concrete has good workability, it is often possible to maintain tremie embedment of 20

feet or more while still maintaining concrete flow. However, excessive embedment of the tremie into the concrete can cause the reinforcing cage to start to lift along with the rising column of concrete.

If the concrete in the drilled shaft starts to lose workability, concrete will not flow readily out of the pipe and will fill the tremie without emptying. Segregation of the mix within the tremie can also cause a blockage. If workers are observed to shake the tremie from side to side or "yo-yo" the tremie up and down, this action is usually in response to a problem with flow out of the tremie. Flow can sometimes be re-established by such action, but often the re-established flow is a result of the concrete forming a "vent" alongside the tremie pipe to the surface of previously placed concrete. Such "volcano flow" is prone to entrap laitance or sediment atop the stiffened concrete as illustrated in Figure 7-12. The long-term solution to this problem is to adjust the mix characteristics (described subsequently in Section 7.5) so that workability is maintained for the duration of placement operations.

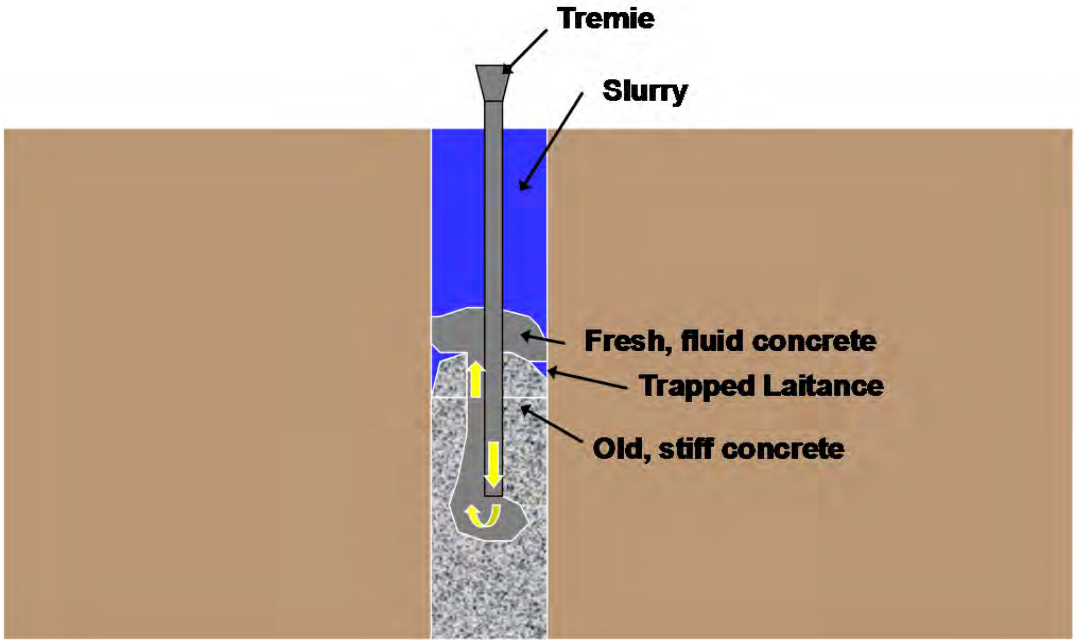
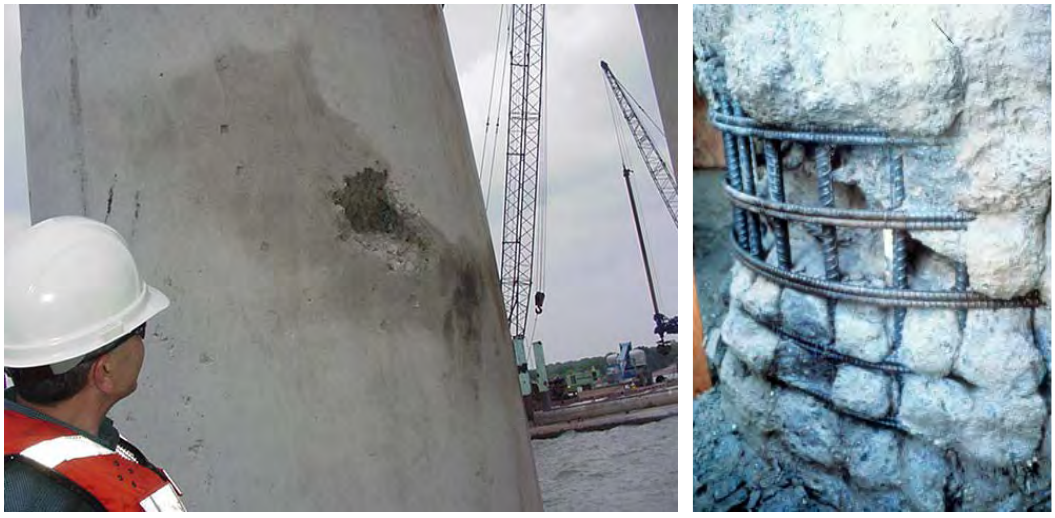


Figure 7-12 Schematic of Concrete "Vent" Due to Loss in Workability During Tremie Concrete Placement

The photos in Figure 7-13 illustrate potential consequences from loss of workability during concrete placement. The photo in Figure 7-13(a) is from a drilled shaft that was constructed using a removable form; the evidence of trapped laitance was the presence of pockets of weak, slightly cemented material on the surface of the drilled shaft. In an exposed location as shown (this column was in a lake), weak material near the surface may pose durability problems and spalling of the concrete cover over the reinforcement. The photo in Figure 7-13(b) is from a drilled shaft that had an interruption in concrete delivery during placement; the delay resulted in a loss of workability in the older concrete. The defect was detected by integrity testing and subsequently repaired (see Chapters 16 and 17).



(a) (b)

Figure 7-13 Effects of Loss in Concrete Mix Workability During Tremie Placement: (a) Trapped Laitance on Exposed Surface; (b) Effect of Interruption of Concrete Delivery

7.3.3.2 Placement by Pump

Concrete pumps are often used to deliver concrete from a convenient discharge location for ready-mix trucks to the gravity-fed tremie. However, a closed pump tremie system may also be used in lieu of a gravity tremie for underwater placement of concrete into the shaft itself. The basic principles of underwater placement are identical to the gravity tremie method, only the delivery system is slightly different. The pump line within the shaft is typically a rigid steel pipe, 4 to 6 inches diameter, which is connected to the delivery line via a short section of flexible hose, as shown in Figure 7-14. Clamp-type connectors include rubber seals to maintain watertight joints. An advantage to the closed pump system shown is that the crane can lift the pump line system during concrete placement without the need for a worker at the top of the tremie to direct flow and manage placement operations.

General issues described previously relating to starting concrete flow and developing a head of concrete with a gravity tremie also apply to a pumped closed tremie. Due to the relatively small diameter of the pump line, a lean cement mix or commercially supplied product is typically used to lubricate the line just prior to pumping concrete. Flexible pump lines have been used to place concrete within a shaft on occasion, but are more difficult to keep straight within the excavation and more difficult to hold close to the bottom of the excavation and control during the initiation of concrete flow. Rigid tremie pipes are therefore preferred for pumped concrete operations. It is also essential that the pump be capable of delivering concrete in sufficient volume to keep up with the flow out of the line, or else the siphoning effect of the gravity flow of concrete down the line could lead to cavitation in the line and potential segregation of the mix (Gerwick, 1987).

As with a gravity tremie, the pumped tremie line should be maintained with a minimum 10-ft embedment into the concrete, but the line should be lifted as the column of concrete within the shaft rises. Even if concrete is flowing, holding the line to the bottom of the shaft can lead to upward displacement of the reinforcing cage.

Concrete placed with a pump line system must have similar workability characteristics as for concrete used with gravity tremie placement. With a closed pump line system, one may be tempted to imagine that the pressure applied to the concrete delivered down the tremie would make it easier to maintain embedment of the tremie line into the fresh concrete during placement. However, if loss of concrete workability occurs as described in the preceding section, the pressure behind the pumpline will tend to make it push up and out of the concrete, and workers will find it difficult to hold the line down. In such cases, defects in concrete integrity could be expected; a closed pump tremie system is not a panacea for problems with a gravity tremie.

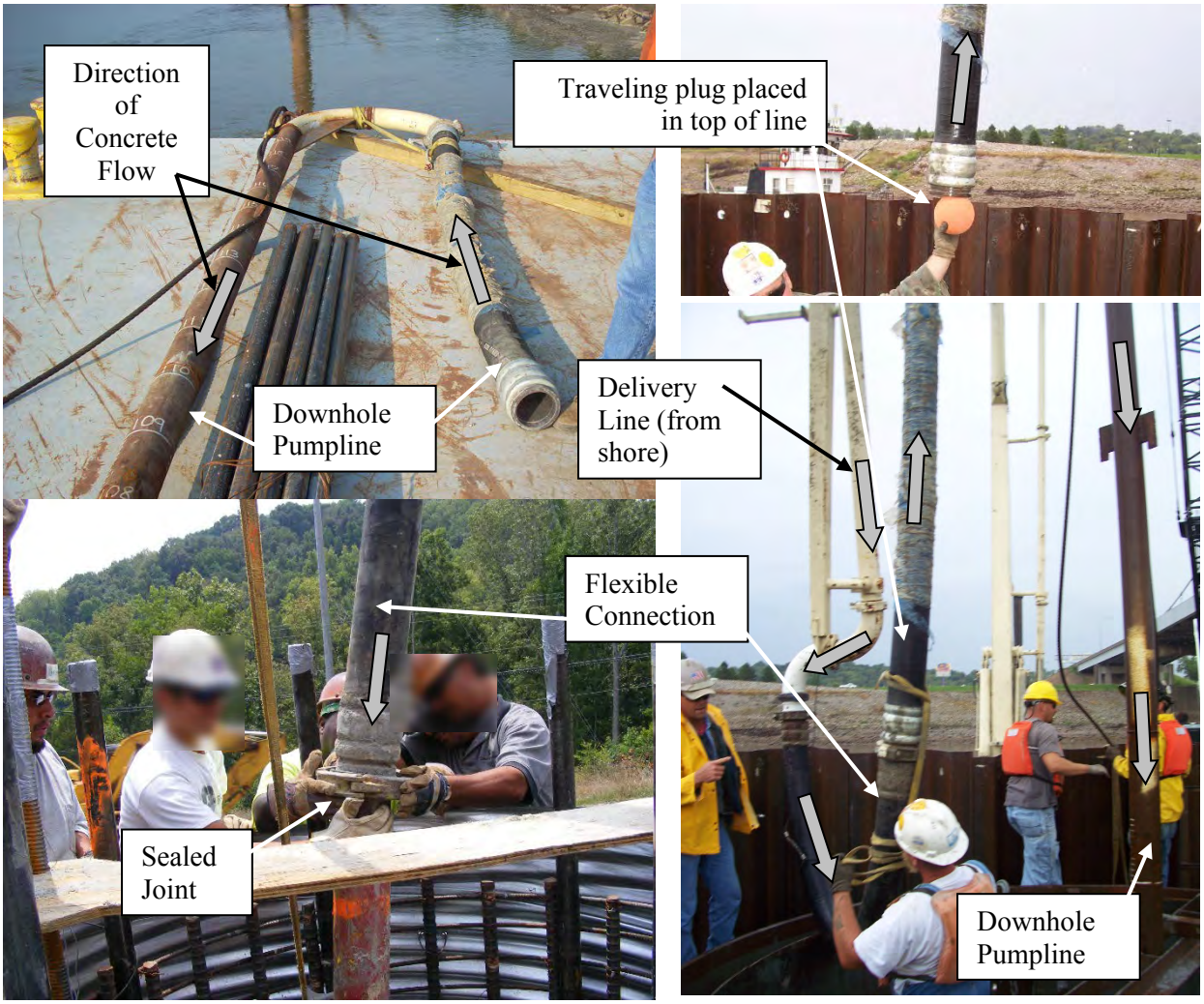


Figure 7-14 Pump Line Operations for Underwater Concrete Placement

7.3.3.3 Importance of Slurry Properties and Bottom Cleanliness

Even with excellent tremie operations and a properly designed concrete mix, there can be defects within the concrete if the excavation is not clean or the slurry is heavily laden with sand. Even if the design of the drilled shaft does not rely on tip resistance (such as a shaft used for a wall or controlled by lateral loading considerations), it is essential that the base of the excavation be reasonably free of loose debris that can be stirred up by the initial charge of concrete from the tremie. Such debris could find its way to

the top of the rising column of concrete only to be folded into the concrete as a trapped inclusion at some subsequent time. In general, no more than 3 inches of loose sediment should be present on the bottom of the shaft prior to concrete placement in order to reduce the risk of soil inclusions; more restrictive bottom sediment requirements are typically used for drilled shafts that rely wholly or partially on tip resistance. The amount of loose sediment will be less for drilled shafts that rely on end bearing resistance for a significant portion of the axial resistance.

The drilled shaft in Figure 7-15 was an experimental shaft that was exhumed for examination. This shaft was cleaned using a cleanout bucket and airlift (as described in Chapter 4) and revealed good quality concrete across the majority of the bottom surface. The perimeter of the shaft base exhibited evidence of a small amount of debris, which tended to be displaced to the outside edge of the excavation by the concrete from the tremie.



Figure 7-15 Exposed Bottom Surface of an Exhumed Drilled Shaft

If sand settles out of suspension during concrete placement, the accumulation of this debris atop the rising concrete column could result in entrapment of pockets of sand within the concrete. The slurry-filled hole is like a giant hydrometer test, with sand slowly settling to the bottom. If sand is observed atop the concrete at the completion of concrete placement, this signal should be a warning that the slurry was not sufficiently clean. The rising column of concrete does not rise as a static, horizontal surface; the surface of the concrete tends to roll from the center near the tremie toward the perimeter, and debris on the surface may tend to become lodged in pockets around the reinforcing and subsequently enveloped in the concrete. For this reason, the sand content in the slurry immediately prior to concrete placement is an important property that must be controlled, as described in Chapter 5. Since polymer slurry does not tend to keep sand or silt in suspension, a lower sediment content must be specified when polymer slurry is used. The sand content guidelines provided in Chapter 5 and in the guide specifications in Chapter 14 are for routine construction. Deep and/or large diameter drilled shafts that require longer times to complete concrete placement may require that the sand content in the slurry be even lower than typically specified. In large or deep shafts, it may be necessary to fully exchange the entire volume of slurry by pumping the old dirty slurry from the bottom of the shaft to a tank or desanding unit, and simultaneously pumping fresh, clean slurry into the excavation.

7.3.4 Completion of Concrete Placement at Shaft Head

At the completion of concrete placement using a tremie, there is often up to several feet of contaminated concrete that must be removed. Even with a dry excavation, there can be contamination at the top of the concrete due to accumulated bleed water. If the top of shaft is at or near the ground surface, this concrete can readily be shoveled off by workers until clean fresh concrete is revealed, as shown in the photo of Figure 7-16.



Figure 7-16 Over-pour of the Shaft Top Until Clean Concrete is Revealed

If the design includes a finished top of shaft elevation that is below grade, completion of the shaft is more difficult. In most cases, the contractor will prefer to over-pour the drilled shaft and remove slurry and contaminated concrete before it hardens. This operation requires a casing to be left in place if the top of shaft is more than a few feet below grade. In some cases with deep top of shaft cutoff, the contractor may be forced to remove contaminated material after the concrete hardens using jackhammers. Deep cutoff elevations (more than 20 ft) may require additional safety considerations for worker entry into the hole.

7.4 DRILLING NEAR A RECENTLY CONCRETED SHAFT

The guide specification provided in Appendix D includes a requirement that concrete in a recently concreted shaft must achieve an initial set before drilling or casing installation can be done in the vicinity. The definition of "vicinity" in this specification is defined as a clear spacing of three shaft diameters. The most important reason for this specification is the possibility of communication between nearby excavations, wherein the fluid pressure of the fresh concrete breaks through to the nearby excavation. This concern can be particularly acute in karstic regions, where an increased separation distance may be needed. Vibro-driving of casing can also create elevated pore water pressures near fresh concrete which could lead to excessive bleed water or instability of the fluid concrete-filled shaft. There is little evidence of structural damage to the concrete due to normal drilled shaft construction operations.

Studies reported by Bastian (1970) indicate that nearby construction, such as drilling or pile installation, does not normally damage concrete within a recently placed drilled shaft. He reports on a case where pile driving was being done 18 ft away from a shell-pile that had just been filled with fresh concrete. Cores were taken three days after placing the concrete. Subsequent testing showed that the compressive strength of the cores was slightly greater than that of concrete cylinders taken during placement of the concrete. Bastian reports on five other investigations by various agencies and groups. In each of the cases, measurements showed that the properties of fresh concrete were not adversely affected by vibration. Bastian reached the following conclusion: "There is ample evidence that the vibration of concrete during its initial setting period is not detrimental ... It can be concluded, therefore, that vibrations due to the driving of piles immediately adjacent to freshly placed concrete in steel pile shells is not harmful to the concrete and no minimum concreting radius should be established for this reason."

A typical concrete mix used for drilled shafts is expected to achieve a set within 24 hours. Exceptionally long placement times may require a mix with a high dosage of hydration control admixtures that could extend this normal limit. The 24-hour limit can sometimes be shortened by using silica fume or admixtures. Use of high-early-strength cement is generally not recommended if shaft diameters exceed 5 feet because of the high heat of hydration. In general, the contractor must simply manage the construction sequence for closely spaced drilled shafts to meet this requirement in the guide construction specifications.

7.5 CONCRETE MIX DESIGN

Concrete mixes for drilled shaft concrete must achieve the required properties needed for the contractor to build a completed shaft that meets the structural requirements for the project without introducing excessive risk of construction problems. The most important properties for drilled shaft concrete are those associated with the fresh concrete and its impact on construction operations. This issue is illustrated in Figure 6-6, which shows drilled shafts constructed using a concrete mix that did not adequately flow through the rebar cage. Technical demands on the properties of hardened concrete are usually rather ordinary, as the structural stresses in the concrete are typically modest compared with many civil engineering works. Compressive strength requirements of 4,000 to 5,000 psi are typical and easily achieved with modern materials. There have been a few isolated cases where higher strength concrete has been utilized, mostly for high-rise buildings. The sections that follow describe the components and procedures used to develop concrete mix designs that are suitable for drilled shaft construction.

7.5.1 Cementitious Materials

Several cementitious materials are commonly used to develop mix designs that achieve the often competing performance requirements for drilled shaft concrete. Portland cement is the most common cementitious material in drilled shaft concrete, although some proportion of the Portland cement may be replaced with supplementary cementitious materials (SCMs) to improve characteristics of the concrete as described in the following sections.

7.5.1.1 Portland Cement

Portland cement for drilled shafts should meet the requirements of ASTM C 150 (2017). Type I or Type I/II Portland cement is normally used for drilled shaft concrete. Type III (high-early-strength) cement should usually be avoided, especially in shafts with diameters greater than 5 feet. Type IV (low heat of

hydration) cement may be considered for large diameter shafts where concerns about high in-place temperatures and large thermal gradients exist, but such concerns are more commonly addressed in other ways as described in Section 7.6.5. Type II (moderate sulfate resistance) and Type V (high sulfate resistance) cements, which have limited amounts of tricalcium aluminate (C_3A), should be considered in environments where the sulfate content of the soil or groundwater is high. Availability of Type V cement may be limited, but increased sulfate resistance can also be achieved by using SCMs and appropriately selecting a sufficiently low water-to-cementitious materials ratio (w/cm).

7.5.1.2 Supplementary Cementitious Materials

Addition of SCMs to ordinary Portland cement, or replacement of a portion of the Portland cement with SCMs, may improve the characteristics of fresh and hardened drilled shaft concrete. SCMs such as fly ash, slag cement, and silica fume are industrial by-products that are commonly used in concrete. Class F fly ash (ASTM C 618, 2017) and silica fume are pozzolanic materials that possess little strength when hydrated by water, but form cementing materials in the presence of Portland cement, particularly the free calcium hydroxide (lime) that exists in Portland cement. Class C fly ash (ASTM C 618, 2017) is typically high in calcium oxide content (CaO > 20%) and exhibits both hydraulic (similar to Portland cement) and pozzolanic behavior. Because of this characteristic, the rate of strength development of concrete mixtures made with Class C fly ash is typically more rapid than concrete made with Class F fly ash. Both Class F and Class C fly ash are commonly used in the concrete industry; however, local availability is determined by the nature of the coal burned in nearby power plants. Class F fly ash may contain some unburned carbon (as shown by high loss on ignition, or LOI values) that causes difficulties in consistently entraining sufficient amounts of air to obtain concrete that is resistant to cycles of freezing and thawing.

Slag cement, also known as ground-granulated blast-furnace slag, is a by-product from iron production. When used in concrete, slag cement is commonly proportioned to replace 30 to 50 percent of Portland cement by weight. Unlike fly ash, which can be used directly after collection from the stack of a power plant, slag cement must be ground to the desired fineness before it can be used as a cementitious material. In the United States, slag cement is specified according to ASTM C 989 (2017), which provides for three grades of slag cement depending on its reactivity index. The reactivity index provides a measure of the relative 28-day mortar strength obtained from a 50-50 slag cement-Portland cement blend relative to the strength of pure Portland cement mortar. The classifications are Grades 80, 100 and 120, with higher grades indicating greater reactivity index. Slag cement is a latent-hydraulic material that in the presence of hydroxyl ions reacts similar to Portland cement. Slag cement particles are usually ground to be slightly finer than Type I Portland cement. Hydration of slag cement is more sensitive to curing temperature than Portland cement. In cold weather, the rate of hydration of slag cement will be significantly retarded, which can result in extended setting times.

Large dosages of fly ash or slag cement tend to slow strength development compared to mixtures with only Portland cement. However, when cured, these mixtures may exhibit greater long-term strength. It is therefore advisable to test the compressive strength for concrete mixes with large dosages of fly ash or slag cement at 56 or 91 days in lieu of the 28 days normally specified for conventional concrete. Mixtures containing combinations of fly ash and slag cement have also been successfully used.

Silica fume, which is rich in silicon dioxide, combines with excess lime in Portland cement and produces a cement paste that is usually stronger and more dense than paste produced using either Portland cement alone or using other SCMs. Silica fume consists of very fine particles (100 times finer than Portland cement). The high surface area of silica fume means that it reacts at early stages and typically increases early-age and long-term strengths, and decreases long-term permeability (ACI 234R, 2000). Because of

its high surface area, silica fume significantly increases the water demand of the mixture, which necessitates use of high-range water reducing admixtures to obtain the desired degree of workability. Silica fume is also relatively expensive compared to fly ash or slag cement.

One advantage of using SCMs in Portland cement concrete is that they convert calcium hydroxide to calcium silicate hydrate over time, which is the most dense and desirable product produced when Portland cement hydrates. The formation of increased amounts of calcium silicate hydrate reduces pore space in the concrete, which leads to greater long-term strength, reduced long-term permeability, and a general improvement in long-term durability. Fly ash and silica fume also generally reduce the amount of bleeding experienced by concrete, which can be problematic if bleed water escapes through channels in the concrete or at interfaces between concrete and reinforcement.

SCMs also produce other desirable effects in drilled shaft concrete. Fly ash and slag cement tend to reduce the heat of hydration, which may be important for large-diameter shafts. SCMs also tend to retard the set of the cement paste, thereby increasing the time that the concrete remains workable. Use of fly ash will also reduce the water demand of a mixture because of its spherical particles. Conversely, use of slag cement will tend to increase water demand because of its increased fineness and angular particle shape. Silica fume is typically used to obtain very low permeability concrete for projects that require extended design lives. However, use of silica fume will adversely affect concrete workability, and concrete experts should be consulted when it is used in drilled shaft applications. Table 7-1 shows typical Portland cement replacement rates for some supplementary cementitious materials that should be considered for drilled shaft concrete. These amounts vary from location to location since the quality of both the cement and SCMs vary. Table 7-1 should therefore be considered only a general guide.

TABLE 7-1 TYPICAL PROPORTIONS OF POZZOLANIC ADDITIVES

Type of Supplementary Cementing Material	Cement Replacement Dosage (percent of total cementitious material by weight)
Class C Fly Ash	20 to 30 %
Class F Fly Ash	15 to 25 %
Slag Cement	30 to 50 %
Silica Fume	5 to 8 %

Note: Ternary mixtures made with both fly ash and slag cement typically contain up to 20% fly ash and 30% slag cement by weight of total cementitious materials.

7.5.2 Aggregate and Water

Besides cementitious materials and admixtures, the materials used in concrete consist of aggregates and mixing water. Natural materials are normally used as aggregates for drilled shaft concrete. Lightweight aggregates are not ordinarily recommended, as there are no benefits to using this type of aggregate in drilled shafts. Natural aggregates (natural gravel, sand and crushed stone) used for drilled shaft concrete in the United States are typically stronger and less permeable than the hydrated cement paste in the hardened concrete. For this reason, conventional wisdom states that the largest aggregate should be as large as possible and that the aggregate should be well-graded in order to minimize the amount of paste in the mix. However, the benefits of increased size of coarse aggregate is offset by the need for the concrete to flow freely through openings in the reinforcing cage.

Aggregate size generally controls the passing ability of concrete through the reinforcement, as discussed in Chapter 6. As such, relatively small coarse aggregate should be used for drilled shaft concrete mixes. A nominal maximum aggregate size of 3/4 inch has performed well in routine applications involving dry placement operations without congested reinforcement. Smaller maximum aggregate sizes should be considered in drilled shafts where the clear spacing between reinforcing bars is restricted and/or tremie placement under fluid is anticipated. Use of a nominal maximum size of 1/2 inch has successfully been used in congested applications (Brown, et al, 2007). Many agencies now routinely use 3/8-inch nominal maximum aggregate size for drilled shaft concrete because of concerns for congested reinforcement and tremie placement operations. Aggregate aspect ratio (i.e., length to width) should also be considered for crushed aggregates, because oblong-shaped aggregates do not flow as well as more spherical particles.

Aggregate for drilled shaft concrete should generally be well-graded and durable. All aggregate should meet relevant specifications of the American Society for Testing and Materials (ASTM) for concrete aggregate. These specifications include ASTM C 33 (2016) *Standard Specification for Concrete Aggregates*; ASTM C 87 (2017), *Standard Test Method for Effect of Organic Impurities in Fine Aggregate on Strength of Mortar*; and ASTM C 227 (2010), ASTM C 289 (2007), ASTM C 295 (2012), and ASTM C 586 (2011), all of which address tests that measure the susceptibility of aggregates to alkali silica reactivity (ASR).

Water used for the concrete need not be potable (free of organic contamination and deleterious materials); however, almost any natural water that is drinkable can be used for making concrete (Kosmatka and Wilson, 2016). Water for concrete should have low chloride and sulfate contents. Questionable water can be assessed by comparing the effect of its use on setting times and 7-day compressive strength (Kosmatka and Wilson, 2016), although these tests would not address questions related to long term effects such as durability.

7.5.3 Chemical Admixtures

Chemical admixtures are often used for drilled shaft concrete because of the challenging requirements described previously in this chapter. While chemical admixtures provide important capability for meeting these requirements, successful use of chemical admixtures depends on many factors, including the methods of preparation and batching, ambient temperatures, and sometimes complex interaction between different admixtures. Improper dispensing of chemical admixtures can lead to erratic and often undesirable concrete behavior. Chemical admixture companies should therefore be consulted to determine the most suitable admixture type(s) and dosage for the specific project, materials, and placement conditions. Dispensing of admixtures must also be carefully controlled and routinely inspected to ensure that correct amounts are added to each batch. The reader is referred to Section 1.7 of ACI 212.3R (2004) for more information on preparation and batching of chemical admixtures. Admixtures are covered by various ACI and ASTM specifications, some of which are described individually below.

7.5.3.1 Air-entraining Admixtures

Air-entraining admixtures are used to introduce and stabilize entrained air bubbles in concrete. Entrained air is commonly used to improve concrete resistance to freeze-thaw damage. While not necessary for many drilled shaft applications, air-entraining admixtures (ASTM C 260, 2016) can be used in drilled shaft concrete when deterioration of the concrete by freeze-thaw action is possible (e.g., for secant pile walls where shafts will be exposed). Entrained air also generally improves workability and reduces

bleeding, but increases the porosity of concrete which reduces strength (ACI 212.3R, 2004). Entrained air can also improve a mixture's pumpability when at least 5 percent air is produced in a mix.

Despite the potential benefits of air entrainment, it is important to recognize that entrained air may be lost when subjected to substantial hydrostatic pressure, such as will be encountered for deep shafts. Loss of entrained air will reduce workability and produce a situation where the workability observed in tests performed at the ground surface is not representative of the workability at depth. Entrained air should therefore be used only when truly needed to produce freeze-thaw resistance, and its effect on workability should be appropriately considered when establishing acceptable ranges for workability.

7.5.3.2 Set-Controlling Admixtures

Set-controlling admixtures (also known as retarding or hydration stabilizing admixtures) are often used in drilled shaft concrete to provide adequate time to complete concrete placement. Retarders (ASTM C 494, 2017) consist of lignin, borax, sugars, tartaric acids and salts. These admixtures are frequently required for large placements and when concrete is placed during periods of high temperatures (> 80°F). While it is important to retard the set of concrete in many field settings, use of excessive retarders can keep concrete fluid for too long and affect long-term strength.

Hydration stabilizing admixtures (also known as extended-set control admixtures) act to stabilize the hydration of Portland cement, delaying it from reaching initial set. Once the effect of the admixture wears off, hydration of the cement will resume without sacrificing the hardened properties of the concrete. Hydration stabilizing admixtures have been successfully used for drilled shafts on many projects and, in many cases, are more effective for controlling the set time of drilled shaft concrete than conventional retarding admixtures. Most major chemical admixture suppliers have hydration stabilizing admixtures, which are classified as a Type D Retarder by ASTM C 494 (2017).

It is important that the dosage of set-control admixtures be established based on the ambient temperature conditions and concrete temperature expected at placement. The haul time (including some allowance for traffic delays) from the concrete plant to the construction site should be estimated and used to determine the most suitable admixture type and dosage. Higher dosages of set-control admixture will be required under warm weather placement conditions. The required admixture dosage should be determined by trial placements where the conditions expected during construction are replicated, and a sample of concrete is retained and tested to determine its workability retention characteristics.

7.5.3.3 Accelerating Admixtures

An accelerating admixture is "an admixture that causes an increase in the rate of hydration of the hydraulic cement and thus shortens the time of setting, increases the rate of strength development, or both" (ACI 116R, 2005). Accelerating admixtures have a place in some instances in substructure and superstructure construction, but they should not be used in drilled shaft construction except in extraordinary situations. For example, an accelerating admixture might be appropriately used to minimize washout of cement when a segment of a drilled shaft is being placed in a stratum having rapidly flowing groundwater and a casing cannot be used to seal off the stratum. Concrete specialists should be consulted whenever the use of accelerators is contemplated, and contractors must be attentive to cleaning casings, pumps, pump lines and tremie pipes quickly, before setting occurs.

7.5.3.4 Water Reducing Admixtures

Water-reducing admixtures reduce the water requirements of a concrete mixture to achieve a given workability. Water reducers reduce the surface tension of the water surrounding the cement particles before the concrete begins to set, thereby increasing the workability of the fluid concrete without the need for excessive water. Three main types of water reducing admixtures are commonly used and classified in accordance with ASTM C 494 (2017):

- Low-range water reducing admixture (a.k.a. water reducers): Typical water reduction is around 5% to 10%.
- Mid-range water reducing admixture: Typical water reduction is from 6 to 12%.
- High-range water reducing admixture (a.k.a. superplasticizers): A water reduction from 12% to 30% can be achieved.

To achieve the workability that is desirable for drilled shaft construction without water reducers, the water/cementitious material ratio (w/cm) generally needs to be in the range of 0.50 to 0.60 (by weight). In this context, "cementitious materials" are considered to include the Portland cement and SCMs such as fly ash or slag cement that are made part of the mix. More than half of the water in such mixtures is present only for lubrication of the cement paste during concrete placement and is not needed for cement hydration. The excess water produces a hydrated cement paste that contains many pores, which results in a relatively permeable, weak concrete. With water reducers, the w/cm can be reduced conveniently to 0.45 or lower, which helps produce a more dense and less permeable paste while at the same time providing excellent workability. Both low-range and high-range water reducers have been used in drilled shaft concrete. Low-range water reducers that include lignosulfonates and hydroxylated carboxylic acids (ACI 212, 2004) can be used to obtain workable concrete with w/cm in the range of 0.40 to 0.45. With high-range water reducing (HRWR) admixtures, w/cm can be reduced to 0.35 or less while still maintaining good workability.

First (naphthalene-based) and second (melamine-based) generation HRWRs often experienced rapid loss of workability or “flash set,” which is undesirable. Third generation HRWRs are synthetic materials often called polycarboxylates that are designed to be compatible with regional cements. When used in combination with set-controlling admixtures, polycarboxlylate-based HRWRs have the ability to maintain their level of water-reduction even under hot weather conditions.

Although the lower w/cm achievable with HRWRs will result in a more durable and stronger concrete, use of inappropriate HRWR admixtures can occasionally cause rapid setting, which can be detrimental to drilled shaft construction since the concrete may begin to set if unexpected delays occur. Highly fluid concrete with a slow rate of slump loss is preferred to highly fluid concrete that has a very low value of w/cm but also the potential for undergoing rapid set, even though the final product may not be quite as strong or durable. HRWR admixtures should not be disallowed, but their use mandates site- and material-specific measurements of the slump loss characteristics be made prior to being used for production shafts. Trials batches for evaluating slump loss should be made with the specific admixture that is to be used for the project, and tests should be performed at the ambient temperature at which the concrete is to be placed in order to verify that the admixture does not produce undesirable slump loss. A plan for management of concrete mixtures with HRWRs should also be required as part of the construction specifications (e.g., TxDOT, 2004)

7.5.3.5 Viscosity-Modifying Admixtures

Viscosity-modifying admixtures, or VMAs (a.k.a. anti-washout admixtures), are often used for highly workable mixes (e.g., "tremie concrete" as described in Section 7.5.4) to reduce the potential for segregation and bleeding. VMAs are typically formulated from cellulose ether, welan gum or polyethylene glycol. VMAs improve the performance of tremie concrete mixtures by binding some of the free water, thereby increasing the cohesiveness and viscosity of the concrete, which helps to control segregation and reduce bleeding. VMAs also help to reduce the variability of the fresh properties of tremie concrete mixtures that can arise from variations in free water and placement conditions.

While VMAs can produce desirable characteristics for fresh concrete, they should be carefully used. The most significant concern is that VMAs can make concrete exhibit thixotropic behavior wherein the concrete will stiffen up when stationary, but become more fluid when displaced or disturbed. This behavior can produce a condition wherein the concrete appears to be set, but becomes more fluid if displaced or disturbed. This response can affect flow from the tremie or pump line if placement is interrupted, which can lead to eruptive "volcano flow," as shown in Figure 7-12, and increased potential for trapping laitance. This behavior can also cause problems for removal of temporary casing, particularly segmental casing that requires intermittent interruption of concrete placement operations. VMAs tend to work well as long as delays are avoided and the concrete is kept moving throughout placement.

7.5.3.6 Other Admixtures

Other types of chemical admixtures are available for special cases. Examples of these are anti-bacterial and anti-fungal admixtures, alkali-reactivity reducers, corrosion inhibitors, and pumping aids. Except for pumping aids, which are normally polymer products added to the concrete prior to pumping to aid in lubricating the pump lines, these are rarely used in drilled shaft construction. However, readers should be aware of their existence for special circumstances.

7.5.4 Mixture Proportions

The challenge of mix design for drilled shaft concrete is to establish proportions of cement, water, SCMs, aggregate, and admixtures that will satisfy the challenging and often competing performance requirements necessary for successful construction, and to do so economically. It is important to emphasize that there is no single mix design that will be suitable for use with all drilled shafts in all locations at all times. Appropriate mix designs should therefore generally be established on a project specific basis using the trial mixture design method (e.g., Kosmatka and Wilson, 2016). Trial batches can be developed under laboratory conditions or by a concrete batch plant. Initially, many trial batches of a proposed mixture may be made, with variations to bracket the range of variables such as cement content, SCM dosage, chemical admixture dosage, etc. For large projects, it is not uncommon to evaluate more than thirty mixtures before a final mixture(s) is selected. It is also common practice to develop trial mixtures at three different w/cm values in order to allow interpolation to obtain the w/cm that produces a concrete mixture that satisfy all requirements for a project. Trial mixture testing, and evaluation of that testing, should be carried out by a qualified concrete laboratory. Care should be taken to verify that the materials used and conditions present for trial mixture studies continue to exist during construction. If materials or conditions change (ambient temperature, aggregate source, cement source, SCM type or dosage, type of chemical admixture, etc.), new trial mixture studies should be conducted to ensure that the desired behavior will be achieved.

During trial batching, slump or slump flow should be measured over time to characterize the workability of the fresh concrete and how that workability changes with time. Fresh concrete temperature and total air content (if required) should also be measured and cylinder samples should be prepared for later strength testing. For tremie concrete mixtures, mixture stability should also be evaluated using VSI tests (ASTM C 1611, 2005) and static segregation tests using the column technique (ASTM C 1610, 2006). This static segregation test is important to perform for drilled shaft concretes that have extended setting times and are subject to the effects of gravity that may cause vertical segregation in the shaft. From the static segregation test a percent static segregation is obtained, which should not exceed 10% (ACI 237, 2005).

For large projects, the final step of trial batching should include construction of a full-scale technique shaft to evaluate the performance of the selected mixture when batched in the concrete producer's batch plant and placed by the project contractor. Where possible, conditions for full-scale technique shafts should reproduce the level of rebar congestion of production shafts, the shaft size, placement temperature conditions, and conservative estimates of the haul time including potential delays. During installation of the full-scale technique shaft, the placement characteristics, fresh concrete properties, strength, permeability, and any other project requirements should be assessed.

The availability and use of chemical admixtures often enable the challenging and competing performance requirements for drilled shaft concrete to be effectively met. However, the performance of chemical admixtures can vary substantially with small changes in dosage, with changes in ambient conditions, and changes in other concrete constituents. Additionally, complex interaction among different admixtures can sometimes lead to unexpected performance. As such, it is generally desirable to use mixes without chemical admixtures if such mixes can meet the performance requirements for a project, or to limit use of admixtures to the extent possible given available materials and project performance requirements.

Several example mixture proportions for drilled shaft concrete are shown in Table 7-2 and Table 7-3. The proportions shown in Table 7-2 represent conventional drilled shaft concrete mixes that have produced cohesive concrete mixtures appropriate for dry placement techniques. The proportions shown in Table 7-3 represent tremie concrete mixes that have been used successfully for wet placement in drilled shaft applications. The most appropriate mixture for a specific project will depend on the characteristics of local materials, climate, and project specific requirements (placement method, duration of placement, required passing ability, durability requirements, SCM availability, etc.). The most appropriate chemical admixtures and dosages for a mixture should be selected by the concrete producer with input from the chemical admixture suppliers, and verified by trial mixes and application in technique shafts.

For small projects involving construction of a limited number of ordinary drilled shafts (e.g., small shafts, without congested reinforcement) at sites that do not impose special demands for delivery or placement, developing project-specific mix designs using the trial-batch method may not be practical. For such projects, agencies may develop one or more "standard mixes" that are appropriate for small projects. In most jurisdictions, agencies should generally develop multiple standard mixes. For example, an agency might develop a standard concrete mix design suitable for dry placement methods and a separate standard mix design for tremie concrete. Agencies might also develop alternative standard mixes for summer and winter placement. In large jurisdictions, agencies might also develop alternative standard mixes for different regions to address variations in materials and perhaps climate. All standard mix designs should be developed following the trial-mix design process. Standard mix designs should also be periodically re-evaluated to ensure that variations in materials do not produce performance that is unsuitable for quality construction of drilled shafts.

TABLE 7-2 EXAMPLE MIXTURE PROPORTIONS FOR CONVENTIONAL DRILLED SHAFT CONCRETE

Item	Mixture Type			
	Texas DOT	**Kentucky Trans. Cabinet**	**Indiana DOT**	**Colorado DOT**
Target Consistency	6.5 inch Slump	6 inch Slump	7 - 10 inch Slump	7 inch Slump
Type I or II Cement Content, lb/yd^3	423	526	705	488
Class F Fly Ash Content, lb/yd^3	141	132	--	122
Water Content, lb/yd^3	248	291	286	246
No. 67 Coarse Aggregate (3/4" max), lb/yd^3	--	1667	--	1740
Coarse Aggregate (1/2" max), lb/yd^3	1827	--	--	--
AASHTO 8 Coarse Aggregate (3/8" max), lb/yd^3	--	--	1710	--
Fine Aggregate Content, SSD, lb/yd^3	1390	1155	1120	1312
Water-to-Cementitious Materials Ratio	0.44	0.44	0.41	0.40
Sand-to-Total Aggregate Ratio (by volume)	0.44	0.27	0.40	0.43
Extended-Set Control Admixture, oz/cwt	0-5	10	3-5	--
Water Reducing Admixture, oz/cwt	0-15	4	5-15	8.5

TABLE 7-3 EXAMPLE MIXTURE PROPORTIONS FOR TREMIE CONCRETE

Item	Mixture Type			
	Washington DOT	**Mullica River, NJ Turnpike**	**I-35W, Minneapolis**	**Lumber River, SC**
Target Consistency	7 – 9 inch Slump	18-24 inch Slump Flow	24 inch Slump Flow	18-24 inch Slump Flow
Type I or II Cement Content, lb/yd^3	600	526	242	500
Class F Fly Ash Content, lb/yd^3	-	132	108	250
GGBFS (Slag)	181	0	359	0
Water Content, lb/yd^3	315 (max)	267	270	306
No. 67 CA (3/4" max), lb/yd^3	-	0	1330	1071
No. 8 CA (3/8" max), lb/yd^3	1601	1500	360	395
Fine Aggr. (Sand), SSD, lb/yd^3	1262	1363	1350	1366
Water-to-Cementitious Ratio	0.40 (max)	0.405	0.38	0.41
Sand-to-Total Aggr. Ratio (by vol.)	0.45	0.48	0.44	0.48
Extended-Set Control Admixture, oz/cwt	5.1 (max)	4 (Sika)	Unknown	8 (Delvo)
Water Reducing Admixture, oz/cwt	6.4 (max)	7 (Sika 2100 SP)	6 (BASF 7500)	8-12 (Glenium 3030)
VMA, oz/cwt	0	0	6	2

7.5.5 Concrete Batching

The vast majority of concrete for drilled shafts is mixed at dedicated ready-mix plants and transported to the jobsite in ready-mix trucks. However, some projects may be large enough to justify use of an on-site batch plant, especially if the project is remotely located where ready-mix plants are not available. A photo of a worksite batch plant set up for a large bridge project is provided in Figure 7-17. Jobsite batching provides contractors with greater flexibility and control over the schedule and reduces risks from concrete delivery problems that exist for off-site batch plants. Eliminating transport time may also ease challenges with retaining workability for extended periods, which in turn can affect mix designs. Some suppliers can also bring batched dry ingredients to a job site where they are blended and mixed with water only when the contractor is ready to place concrete; however, such on-site mixers typically have limited capacity such that this option is only practical for relatively small shafts.



Figure 7-17 Worksite Concrete Batch Plant Capable of Batching 130 yd^3/hr

7.6 CONCRETE PERFORMANCE REQUIREMENTS

As described in Section 7.2, drilled shaft concrete must generally meet challenging performance requirements that include workability and workability retention, stability, durability, and strength. Each of these requirements is described in greater detail in the following sections, along with common mix design considerations that can be used to achieve the specific requirements.

7.6.1 Workability

Workability is perhaps the most important characteristic of concrete for drilled shafts. Workability is essential because the concrete must self-consolidate and have sufficient passing ability to flow through the reinforcing cage without the use of vibration. It is also critical that the concrete remain workable until the entire placement operation is complete and any temporary casing is removed from the excavation. Workability requirements are particularly challenging when concrete is placed by tremie or by pumping. The following sections describe workability requirements for "conventional drilled shaft concrete" that is placed in a dry hole, generally by free fall, and for "tremie concrete" that should be used when placing concrete through water or slurry by pumping or gravity tremie.

7.6.1.1 Conventional Drilled-Shaft Concrete

Several methods are available for measuring the workability of concrete, but the slump test is used almost exclusively in practice for conventional concrete mixtures. The slump test is not an ideal method for measuring workability; however, no other test has been generally accepted for field use. Concrete for drilled shafts should have a slump of at least 6 inches and no greater than 9 inches when the concrete is placed in a dry hole by free fall. Free fall placement of concrete having high workability, such as that described subsequently for tremie concrete, should be avoided because it can lead to segregation of the concrete mix.

Adequate workability is best achieved using rounded natural aggregate and natural sand. However, crushed stone is being used more frequently as rounded natural aggregate supplies are being depleted. Crushed stone mixtures require either higher paste contents or increased dosages of water-reducing admixtures to attain a degree of workability comparable to mixtures made with rounded aggregate. If crushed stone is used as the aggregate, care must be taken to wash away all of the dust, because dust can consume water that is ordinarily available for lubrication and hydration of the concrete mixture.

7.6.1.2 Tremie Concrete

Tremie concrete is defined as concrete with the ability to achieve sufficient compaction by self-weight when placed by tremie pipe or pumping under submerged conditions (EFFC-DFI, 2018). Tremie concrete used for constructing drilled shafts through water or drilling slurry should have greater workability than conventional concrete used for dry placement methods. Tremie concrete includes mixes that are formally characterized as being "self-consolidating concrete" or SCC, but also includes mixes that may not be considered SCC.

Tremie concrete should generally have a slump of about 9 inches. However, because of the high workability and flowability of tremie concrete mixes, workability is generally better assessed using *slump flow* (ASTM C 1611, 2014) rather than slump. Slump flow is determined by placing the fresh concrete mixture within a conventional slump cone (without rodding) on a nonabsorbent surface, then withdrawing the slump cone and measuring the diameter of the resulting concrete "patty" (ASTM C 1611, 2014). Figure 7-18 illustrates the relationship between slump flow and the yield stress of fresh concrete, which is inversely related to workability (i.e., greater yield stress corresponds to lesser workability). As shown in the figure, slump flow values between about 16 and 22 inches (400 to 550 mm) are generally desirable for tremie concrete for drilled shafts. Greater values of slump flow may sometimes be specified for improved workability, but such mixes are more prone to segregation and bleed so special attention must be paid to provide for adequate mix stability. Conversely, mixes with slump flow less than 16 inches will have

lesser workability, raising concern regarding the mixes ability to adequately flow and fill the excavation when placed in the wet. The passing ability of tremie concrete can also be assessed using the J-Ring test (ASTM C 1621, 2017); however, the clear spacing between vertical dowels for this test is only 1.735 inches, which is too close to be representative of drilled shaft reinforcement.

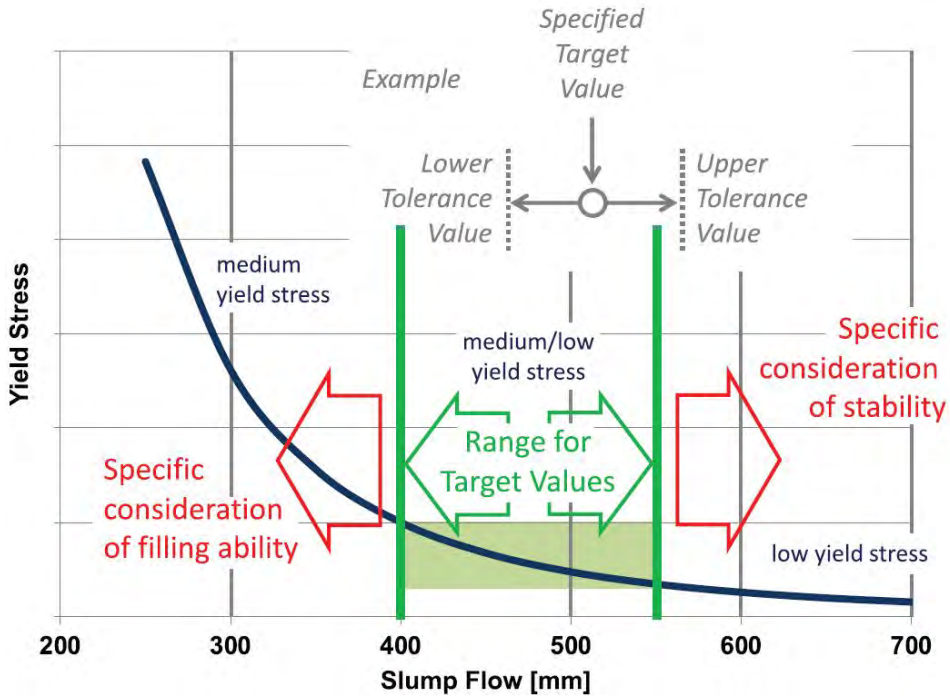


Figure 7-18 Relationship between yield stress (inverse measure of workability) and slump flow (from EFFC-DFI, 2018).

Tremie concrete mixtures developed for drilled shaft construction often utilize a greater sand-to-total aggregate ratio and greater cementitious materials content compared to conventional drilled shaft mixtures. Even though tremie concrete mixtures generally have greater total cementitious material content, the Portland cement content may be lower than conventional mixes due to the use of SCMs. The reduced Portland cement content and the use of SCMs help delay setting and reduce maximum concrete temperatures (Schindler and Folliard, 2005). The greater fines content and use of viscosity modifying admixtures (VMA) produce concrete with high flowability, increased stability (reduced likelihood of segregation of the coarse aggregates), and reduced bleeding (Bailey, et al., 2005).

7.6.1.3 Workability Retention

While workability is a critical requirement for drilled shaft concrete, it is equally critical that workability be maintained for a sufficient period to allow the drilled shaft to be effectively constructed. As a general rule, concrete should remain workable for two hours greater than the estimated time to complete concrete placement and remove the temporary casing, if necessary. When estimating the duration of construction, the contractor should include concrete delivery time, time for extraction of temporary casing, and also allow some additional time for unforeseen delays.

Figure 7-19 illustrates desirable and undesirable workability loss from trial mixture studies for drilled shaft concrete. Desirable workability loss is characterized by slowly decreasing slump or slump flow, and maintaining some minimum value for slump or slump flow for the anticipated duration of concrete

placement. For the example depicted in Figure 7-19, the duration of concrete placement is assumed to be four hours after batching, and it is desired to provide a slump that exceeds 4 inches at that time (a commonly specified requirement). The anticipated duration for concrete placement should consider the time required to transport concrete to the site and the time required to place concrete with reasonable estimates for delays. Four hours is generally a reasonable estimate for routine drilled shaft applications, but substantially greater times may be required for large shafts or projects where concrete delivery is particularly complex (e.g., when concrete must be transported to the site by barge). An undesirable workability loss relationship is also shown in Figure 7-19, in which the initial slump is quite appropriate but slump loss occurs rapidly about 90 minutes after batching. Rapid slump loss can occur when improper dosages of chemical admixtures are used or when the effect of weather conditions are not considered. Without adjusting dosages of chemical retarding admixtures, an increase in temperature of about 18 °F will increase the rate of slump loss by a factor of approximately 2, which means that workability loss measured at 72 °F in the laboratory will be misleading for concrete being placed at 90 °F in the field. Care should therefore be taken to evaluate workability loss at the approximate temperature anticipated for field placement.

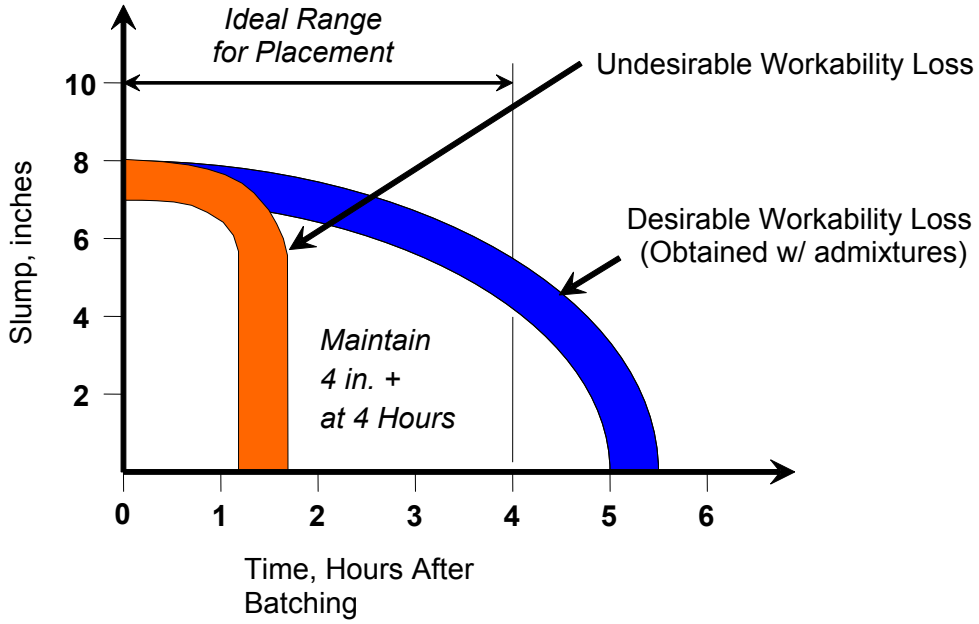


Figure 7-19 Workability Loss Relationship from a Trial Mixture Design

7.6.2 Stability

Concrete mixtures should be proportioned to prevent excessive bleeding or segregation. The potential for bleed and segregation is particularly acute for highly workable mixtures that are commonly used for tremie placement. The potential for segregation and bleeding of a concrete mix can be assessed in the field using the slump flow test by assigning a visual stability index (VSI) to the concrete patty. The VSI is a numerical rating from 0 (indicating no evidence of segregation or bleed) to 3 (indicating clear segregation and bleed), based on the homogeneity of the mixture after the slump flow test has been conducted (ASTM C 1611, 2014). Alternative tests for evaluating the potential for segregation and bleed are also available (EFFC-DFI, 2018), although they are not commonly performed at the present time.

Acceptable limits for bleeding and segregation are a subject of some debate. Sliwinski (1980) defines an acceptable level of bleeding as less than 1 percent of the depth of the pour as long as bleeding does not

occur through channels as shown in the photograph in Figure 7-20. The VSI for acceptable tremie concrete should generally be 1.0 or less. If problems associated with excessive bleed are encountered, the concrete mix should be modified to use less mixing water and/or greater amounts of cementitious materials. Air-entraining admixtures and viscosity modifying admixtures also tend to reduce bleeding.

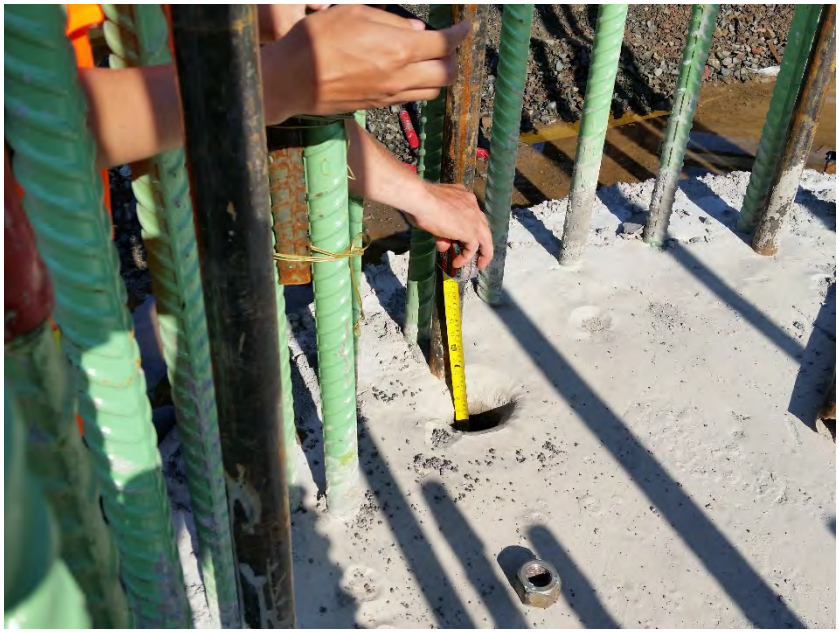


Figure 7-20 Channel formed by excessive bleed water escaping near reinforcing cage.

7.6.3 Durability

The durability of drilled shafts is largely controlled by having an adequate cover of concrete with low permeability outside of the reinforcement. Achieving adequate cover requires that the fresh concrete have adequate passing ability to readily flow through openings in the shaft reinforcement and that the cage is placed in the appropriate position and not excessively displaced during concrete placement. Cage placement should therefore be maintained using appropriate cage centralizers. Concrete passing ability is dictated by the workability and aggregate gradation of the concrete mixtures as well as the spacing of reinforcing bars and spirals/stirrups. Inadequate passing ability can often be identified during placement by measuring the elevation of concrete within and outside of the reinforcing cage. A difference in elevation of concrete inside and outside of the cage of more than a few inches may be indicative of poor quality concrete in the cover zone.

The permeability of concrete is largely controlled by the mix design, primarily the w/cm and the proportion of SCMs that reduce permeability, as described in Section 7.5.1. Concrete permeability can also be degraded by Alkali-Silica Reactivity (ASR), Delayed Ettringite Formation (DEF), or sulfate attack, all which may produce cracking that increases permeability of concrete, particularly in the cover zone. The reader is referred to Chapter 4 of ACI 318 for specific durability requirements for concrete mixtures exposed to various conditions. An aggressive subsurface environment can be considered to exist when the soil, rock or groundwater has free oxygen and/or carbon dioxide (i.e., partially saturated soils), has high concentrations of sulfates or chlorides, or is substantially acidic. Table 4.2.1 of ACI 318 (2008) provides a maximum w/cm limit, a minimum strength, and requirements for the cementitious material composition that depend on the severity of potential exposure to external sources of sulfates. If the water-soluble sulfate in the soils exceeds 0.10 percent by mass or the sulfate (SO_4) in the groundwater (SO_4)

exceeds 150 ppm, the requirements of ACI 318 (2008) should be followed to protect concrete against possible sulfate attack. Some industrial contaminants (usually organic wastes, alkalis and salts) can also be aggressive. Expert assistance should be solicited when industrial contaminants are encountered.

If site conditions are aggressive, consideration should be given to adding SCMs to produce a concrete of reduced permeability. The use of a permanent casing can also help to address concrete durability concerns through aggressive materials.

7.6.4 Strength

The strength of drilled shaft concrete is normally specified by its 28-day compressive strength as tested on 6-inch diameter by 12-inch high cylinders. Most mixtures for drilled shafts will be adequate if they produce 28-day compressive cylinder strengths in the range of 3,500 to 4,000 psi. However, higher strength concrete can be useful under conditions in which the designer wishes to make use of very strong bearing strata and reduce the cross-sectional area of the drilled shaft, which will produce higher compressive stresses in the concrete, or for cases in which high combined bending and axial stresses will be applied to the drilled shaft. Very often, the quantity of cement and SCM used to achieve workability and stability will result in much higher compressive strength values than needed to satisfy design requirements.

Design strengths of 6,000 psi have been used because of seismic design requirements and are achievable with the use of lower w/cm values to produce very low permeability concrete and high-performance concrete. Since the setting time of concrete is reduced when lower w/cm are used, care should be taken to ensure acceptable workability retention behavior, especially under warm weather placement conditions. Higher strength mixtures will also typically contain a greater proportional amount of cementitious materials, which can result in higher in-place temperatures in the concrete during curing.

7.6.5 Control of Concrete Temperatures

Control of temperature is important for drilled shaft concrete because it affects constructability and may affect shaft integrity and durability. Excessive concrete placement temperatures accelerate the rate of hydration and reduce workability. This effect is nonlinear, and rate of hydration increases dramatically with temperature in excess of 70°F. Figure 7-21 shows how initial temperature affects the heat generated within the concrete as a function of time. Generated heat produces more rapid setting in the mixture and a significantly higher heat of hydration in large diameter drilled shafts.

High heat of hydration also introduces potential concerns for drilled shaft integrity and durability, especially for large-diameter shafts that have characteristics of mass concrete in which the heat of hydration can feed on itself and generate large temperatures within the shaft. Two concerns associated with concrete temperature include the potential for delayed ettringite formation, or DEF, and the potential for thermal cracking because of large temperature gradients. DEF causes internal expansion in the cement paste over time and initially results in microcracking that in some instances may progress to severe cracking. Concrete members made with plain Portland cement that reach temperatures above 158°F may experience DEF (Taylor, et al., 2001). The temperature at which DEF may occur increases substantially when Portland cement is replaced with sufficient amounts of fly ash or slag cement, in which case concrete temperatures up to about 170°F can be tolerated without significant concerns for DEF (Folliard et al., 2006). Guidelines for sufficient amounts of SCMs to mitigate against DEF include at least 25% Class F fly ash, at least 35% Class C fly ash, or at least 35% slag cement.

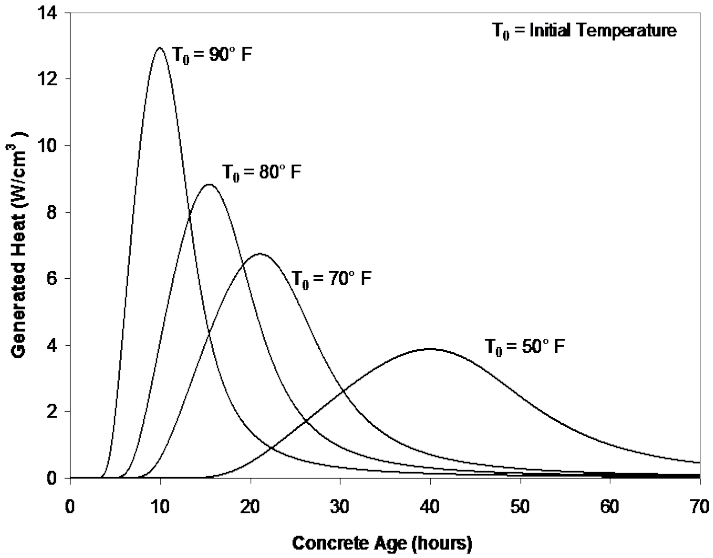


Figure 7-21 Effect of different initial mixture temperatures on temperature development (shown in Watts/cm^3) during adiabatic conditions (Schindler, 2002).

The potential for thermal cracking arises from temperature gradients that develop within a shaft as heat is dissipated to the surrounding material. The magnitude of temperature gradients that develop in a shaft depends on both the temperature generated within the concrete from hydration reactions and the temperature and thermal characteristics of the surrounding material (soil, rock, or fluid). The magnitude of temperature gradients that may produce thermal cracking is a subject of considerable debate. Agency specifications for "mass concrete" often limit the maximum temperature differential to be less than 35°F. However, this guideline is derived from experience with unreinforced concrete in Europe more than 50 years ago (Gajda and VanGeem, 2002) and likely has little relevance to the potential for cracking in concrete that is both reinforced and confined (by soil, rock, or casing).

To the authors' knowledge, there are no known cases of adverse performance of drilled shafts due to thermal cracking or DEF. Nevertheless, measurements made on 10-ft diameter shafts in Florida (Mullins, 2006) have shown temperatures as high as 180°F so the potential for DEF and thermal cracking exists and should be considered when designing concrete mixtures for drilled shafts. The most direct means to mitigate the potential for DEF and thermal cracking is to control the magnitude of concrete temperatures during hydration. In-place temperatures can be controlled by:

1. Limiting the total cementitious materials content.
2. Controlling the fresh concrete placement temperature.
3. Proper selection of cementitious material types.

The amount of total cementitious materials has implications relative to design compressive strength. However, concrete design stresses are often quite low in drilled shafts so it is prudent that the mixture design requirements not exceed the actual performance requirements for design. Because of concerns for setting time and heat of hydration, the use of additional Portland cement to accommodate an unnecessarily high strength requirement can have other implications on mixture performance. Figure 7-21 illustrates the benefits of controlling the fresh concrete placement temperature to control concrete temperature. Temperature controls at the batch plant can be achieved by substituting some of the mixing water with ice, shading aggregate stockpiles, and/or using liquid nitrogen thermal probes to cool the concrete in the truck. Concrete temperatures can also be effectively controlled by using Type II cement

and high dosages of Class F fly ash and/or GGBF slag. Concrete mixtures with high dosages of fly ash or GGBF slag will tend to generate less heat of hydration and are also less prone to DEF.

7.6.6 Communication of Project Specific Requirements for the Concrete

Effective communication is required among all project participants including project owner, designer, contractor, and concrete producer. All too often, specific requirements for drilled shaft concrete are poorly defined during the project bid stage. If all concrete producers do not bid on a mixture that satisfies all project performance requirements, competitive bidding may deliver a producer that either underbid costs associated with the project or may not be able to deliver concrete that meets all performance requirements. Project-specific performance criteria must therefore be clearly defined in project bid documents so that time and cost allowances can be made by the contractor and concrete producer for appropriate testing during the mixture development stage so that a concrete mixture that meets all project requirements can be developed. Routine projects with shaft sizes that allow concrete to be placed in less than 3 hours and routine performance requirements can often be effectively supplied using a "standard" mixture that has "worked well in past projects" without requiring substantial evaluation of concrete mixtures. However, more complex projects involving large diameter or deep shafts, congested reinforcement, complex delivery, or other special challenges can seldom be effectively constructed using standard mix designs and, therefore, will require substantial development and evaluation of concrete mixtures. In all cases, the owner and designer must clearly define and communicate details regarding shaft reinforcement, durability requirements, target service life, and other performance requirements to contractors and concrete suppliers. Contractors must in turn communicate relevant plans that may impact the concrete mix, including anticipated concrete handling and placement methods, estimated duration for concrete transportation and placement, and anticipated ambient temperature conditions (summer, winter, etc.), among others to the concrete producer. The concrete producer must then be prepared to work cooperatively with the contractor to develop a mix that can meet all of the relevant requirements to complete the project successfully.

7.7 SPECIFICATIONS AND QA/QC

Concrete mixtures have historically been specified according to prescriptive specifications that placed relatively narrow limits on mix proportions. While such prescriptive specifications were often acceptable for routine projects, they tended to create costly problems for more challenging projects involving large shafts, remote sites, or other conditions that produced performance demands that exceeded what the prescriptive concrete mixtures could satisfy. To address these problems, many if not most agencies have transitioned to so-called "performance-based" specifications wherein the contractor is responsible for developing a concrete mix design(s) that will meet all project performance requirements while the owner is responsible for performing quality assurance to verify that the constructed drilled shafts achieve all specified performance requirements. While current practices provide the contractor with greater flexibility for achieving project-specific concrete requirements, agencies almost universally still require that selected mix design(s) be submitted for approval prior to being used for construction. As such, the owner retains some control over the composition of concrete mixtures, and this control can sometimes lead to problems, especially for projects with unusually demanding workability and workability retention requirements. In a truly performance-based specification, the owner would relinquish all control over mix design proportions, but instead would require only that constructed drilled shafts satisfy necessary requirements to assure acceptable performance. Current practices should be considered an interim step towards true performance based specifications.

The primary challenge that impedes adoption of truly performance based specifications is that appropriate tests for evaluating drilled shaft dimensions, integrity, and durability have yet to be widely accepted as reliable quality assurance techniques. This challenge is particularly acute for durability requirements, which are currently difficult to verify using independent tests. As a result, durability requirements are largely addressed by constraining mix proportions. Testing and practices for assuring shaft dimensions and shaft integrity are more mature and likely sufficient for evaluating drilled shafts as part of quality assurance. Additional development of methods for assurance of drilled shaft durability is still needed, particularly for projects where structures are required to have a design life of 75 years or more.

7.7.1 Quality Control at the Batch Plant

Materials and operations at the concrete batch plant are important to produce consistent, high quality concrete mixtures that satisfy all project requirements. Items that should be checked at the batch plant include the nature and quantities of the aggregates, cementitious materials, water, and chemical admixtures. There have been occasions when errors have been made at the plant in mixture proportions, with the error not being identified until cylinders are tested at some later date. The consequences of such errors can cause great difficulty and construction delays. Kosmatka and Wilson (2016) state that "specifications generally require that materials be measured for individual batches within the following percentages of accuracy: cementitious materials ±1%, aggregates ±2%, water ±1%, and admixtures ±3%."

It would not be unusual for aggregate characteristics to change as a project progresses, such that a new mix design would be required. Depending on how the aggregates are stored, the moisture content of the aggregate may experience rapid changes with time so that the amount of water to be added to the mixture must frequently be adjusted to ensure that the approved water content is not exceeded in any concrete batch. Accurate control of the water added to each batch is even more critical for mixtures that include HRWA admixtures because such mixtures often have lower w/cm targets. Modern concrete batch plants utilize moisture probes that take real-time measurements of the moisture content of the aggregates to help address moisture variability to provide mixtures with consistent w/cm.

Another important factor in the making of concrete is the temperature of the components of the mixture. For example, hot aggregates and mixing water could produce accelerated setting in concrete during placement. An inspector at the batch plant should check the temperature of the components and the completed mixture for conformance with specifications.

7.7.2 Tests at the Jobsite

Concrete placement should be organized such that the time required to perform tests at the jobsite is practically minimized. There are two reasons: first, the excavation should remain open for as short a period of time as possible to reduce the potential for caving of the excavation and, second, the concrete should be placed as rapidly as possible to maintain workability during the entire placement operation. Because of the first requirement, batch-plant inspection and the timely ordering and delivery of concrete should be emphasized. Delivery of concrete that does not meet the project requirements should be avoided because rejection of delivered concrete can interrupt continuous placement of concrete, which can cause numerous problems that include caving of the borehole, collection of sediment on top of concrete in the shaft, and workability loss that is substantial enough to cause problems with completing drilled shafts construction (e.g., "venting" of fresh concrete over previously placed concrete, displacing the cage, trouble removing temporary casing, etc.). Care must be taken to ensure that concrete is delivered at a pace that the entire pour can be completed without delay and within the workable life of the concrete

mix. Thus, it is essential for the contractor to make an estimate of the as-built size of the excavation to order enough concrete to fill the excavation, allowing for some inevitable losses and overpour. It is also essential that the contractor estimate the time required to complete the construction of the shaft, including contingency for some delay. The time required for construction must be passed along to the concrete producer, which can then add sufficient amounts of retarding admixtures to allow the concrete to remain acceptably workable throughout construction of the shaft.

Jobsite inspection is also essential to ensure that unacceptable concrete does not enter the drilled shaft. Jobsite concrete testing should be viewed as a process of verification and not as a process of control. Recommended minimum jobsite testing should include measuring temperature, slump or slump flow, total air content (if required), and casting cylinder samples for later strength testing. VSI tests (ASTM C 1611, 2005) should also be performed as part of slump flow measurements for tremie concrete mixtures.

Visual inspection of the mix as it enters the hopper is also important. Cemented balls, or zones of segregated or concentrated quantities of aggregate or cement will be evidence of inadequate mixing, and may be cause for rejection of the remaining concrete from the ready-mix truck. Poor flowability through the tremie or pumpline may also be an indication of insufficient concrete workability, or evidence of inappropriate concrete placement procedures.

Agencies have different rules for frequency of sampling. Generally, a set of cylinders consists of two 6 by 12 inch cylinders. At least one set of concrete cylinders should be made and tested per drilled shaft, but not more than one set per truckload for quality assurance. Cylinder samples can be made from small representative samples recovered from each ready-mix truck in a few minutes, freeing the truck to provide concrete for placement in the drilled shaft immediately. It is important that the sampled concrete is representative of the concrete delivered to site; otherwise test results may be misleading. Samples should not be taken from the first or last part of the concrete discharged from the truck. Cylinders should be cured and tested in accordance with project specifications.

Because many projects require placement by gravity tremie or pump, the concrete that first rises to the top of the shaft is normally concrete that was placed first. For large pours, it is therefore recommended to test the workability retention of the mixture by periodically performing slump or slump flow tests from the first load placed in the shaft. The fresh concrete for workability retention evaluation should be sampled at the point of discharge into the tremie or pumpline and stored in a sealed container that is not exposed to direct sun light or vibration. Care should be taken, however, to keep the sampled concrete at similar temperatures to those that exist below ground (often between 55 and 66 °F in the contiguous 48 states); otherwise, slump or slump flow values will not be representative of the condition of the in-place concrete. Prior to performing workability retention tests, the concrete should be slightly agitated by mixing it with a shovel. It is also recommended to overpour the shaft to confirm that good-quality, uncontaminated concrete continues to flow from the borehole to the drilled shaft cut-off level. Where concrete is placed with a drop chute (e. g., in the dry, with or without casing), the first concrete placed will remain on the bottom of the borehole. It is still desirable that this concrete remain workable until the last concrete is placed at the top of the drilled shaft to ensure that ground pressures are reestablished. Workability of concrete in a dry hole is particularly important when temporary casing will be removed during or immediately following concrete placement.

7.7.3 Addition of Water at Jobsite

One of the reasons for rejecting a batch of concrete is that the slump is too low. The question always arises as to the advisability of adding water to concrete in a ready-mix truck. The added water will increase workability, but will have the detrimental effect of reducing the strength and durability of the

concrete. The result of adding water at the jobsite could be a significant change in the characteristics of the mixture and increase the possibility of segregation as the concrete is placed.

In some cases, only part of the mixing water is added at the batch plant, and some of the water is withheld from the approved total mix water with the specific intention to add it at the jobsite. In these cases, the amount of water that can be added at the jobsite should be stated on the mixture batch sheet carried by the ready-mix truck driver. If water is intentionally withheld and the ready-mix truck has a way to accurately measure the volume of water added on site, then additional water can be added without harm, provided the mixing drum is rotated a sufficient number of times to uniformly mix the added water with the concrete mixture. If the slump is then adequate, the placement can begin. In some unusual circumstances such as very long or unpredictable travel times from the batch plant to the jobsite, consideration may be given to bringing dry ingredients to the jobsite and mixing them with water just prior to placement; however, this practice is discouraged since quality control on-site will be inferior to that achieved in the controlled setting of the batch plant.

If all water permitted by the mixture design has been added and the slump is still insufficient, the inspector must note the deficiency and inform the contractor. The contractor is then faced with the decision of adding water sufficient to satisfy workability requirements or ordering new concrete. The decision is a difficult one in situations where additional delay can have other negative consequences. On the other hand, adding water beyond that permitted may risk introducing concrete into the shaft that does not meet project requirements. Where durable concrete of low permeability is essential due to aggressive environments, the addition of excess water may be absolutely disallowed by the contract. Where low permeability is not a major concern, the contractor may choose to proceed at risk, with extra samples obtained for verification testing. If deficiencies are determined to exist, evaluation of concrete quality should be performed and repair techniques implemented, if necessary as discussed in Chapter 17.

7.8 SUMMARY

One of the most critical aspects for drilled shaft foundations is the proper placement of concrete in the excavated borehole to ensure construction of a structurally sound drilled shaft foundation. This chapter has provided a description of concrete placement techniques and equipment covering a broad range of conditions, including a discussion of common problems related to field operations and how to avoid them. The mix characteristics of drilled shaft concrete are also described, with particular emphasis on design of the mix to produce fresh concrete with properties that are needed to achieve constructability. Select examples of successful mix designs are provided, along with recent advances in the development of concrete mixes for challenging drilled shaft construction conditions. Routine tests performed for quality control and quality assurance are also described.

With an appreciation of the important issues discussed in this chapter on drilled shaft concrete and previous chapters on other aspects of drilled shaft construction, an engineer is prepared to consider the design aspects of drilled shaft foundations presented in the following chapters of this manual, and to understand and apply the principles necessary to “design for constructability.”

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CHAPTER 8
DRILLED SHAFT DESIGN PROCESS

8.1 OUTLINE OF THE OVERALL DESIGN AND CONSTRUCTION PROCESS

This chapter provides an overview of the *process* involved in design and construction of drilled shafts. A complete logical process is important because it provides a framework within which an agency, through its planners, designers and project managers, can ensure that all of the elements required for successful design and construction of drilled shafts are identified and addressed. The sequence of steps is not as important as verifying that important issues are not overlooked. It is particularly important that a review of major design issues as well as those issues pertaining to risk identification and constructability be performed early in the process of planning, preliminary design, and foundation type selection. More in-depth treatment of specific design and construction issues is provided in the following chapters of this manual.

The size and complexity of a project will often determine the magnitude of work performed in the planning and preliminary phases. Early phases of the design process may include significant calculations based on preliminary site information which will be revised multiple times, or they may include only rudimentary estimates of drilled shaft performance based on experiences with similar projects. The complete design phase may require numerous iterations in order to achieve an optimal design or to accommodate constructability concerns, or if the project requirements change during the course of the work.

The reliability of a drilled shaft foundation system depends on verification that the design intent is met by the shaft being constructed appropriately for the ground conditions actually encountered. Drilled shafts provide a flexible foundation solution that can allow for adjustments in the field when variable soil and rock conditions may be present. The design is typically based on some criterion of a minimum length of embedment into a designated bearing stratum or geological formation, and the final tip elevations may vary depending on variations of stratigraphy and the soil and rock conditions encountered during construction. A very thorough geotechnical investigation can minimize, but not eliminate, potential deviations from planned tip elevations. The critical aspects of the design must be conveyed to construction and inspection personnel so that the design can be implemented properly and so that any unusual deviations from expected conditions can be identified and addressed appropriately.

The drilled shaft design-construction process is outlined in the flow chart presented in Figure 8-1. This flow chart provides a checklist to guide the designer through all of the tasks that must be completed. Each of these tasks is then discussed using the numbers in the blocks as a reference.

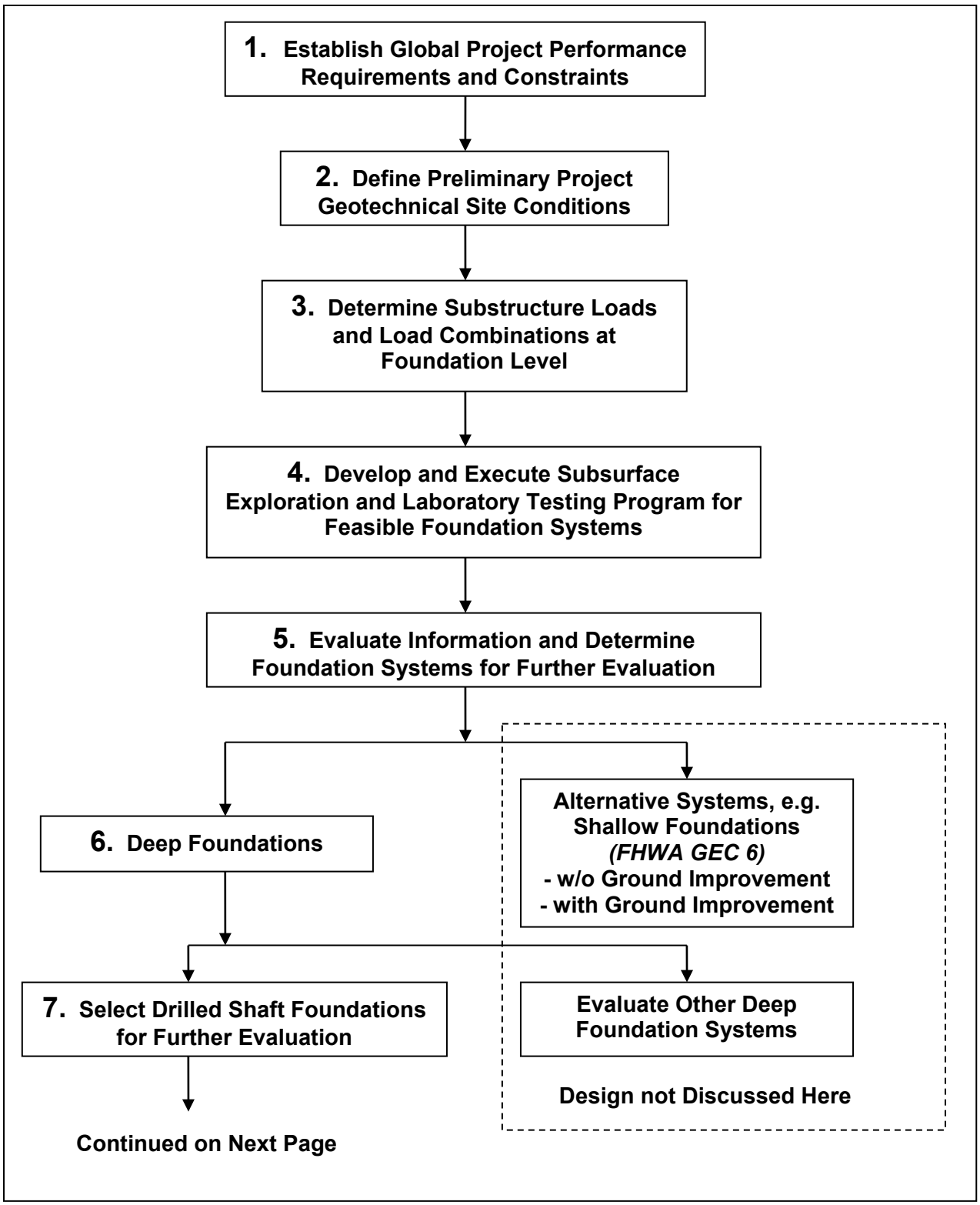


Figure 8-1 Drilled Shaft Design and Construction Process

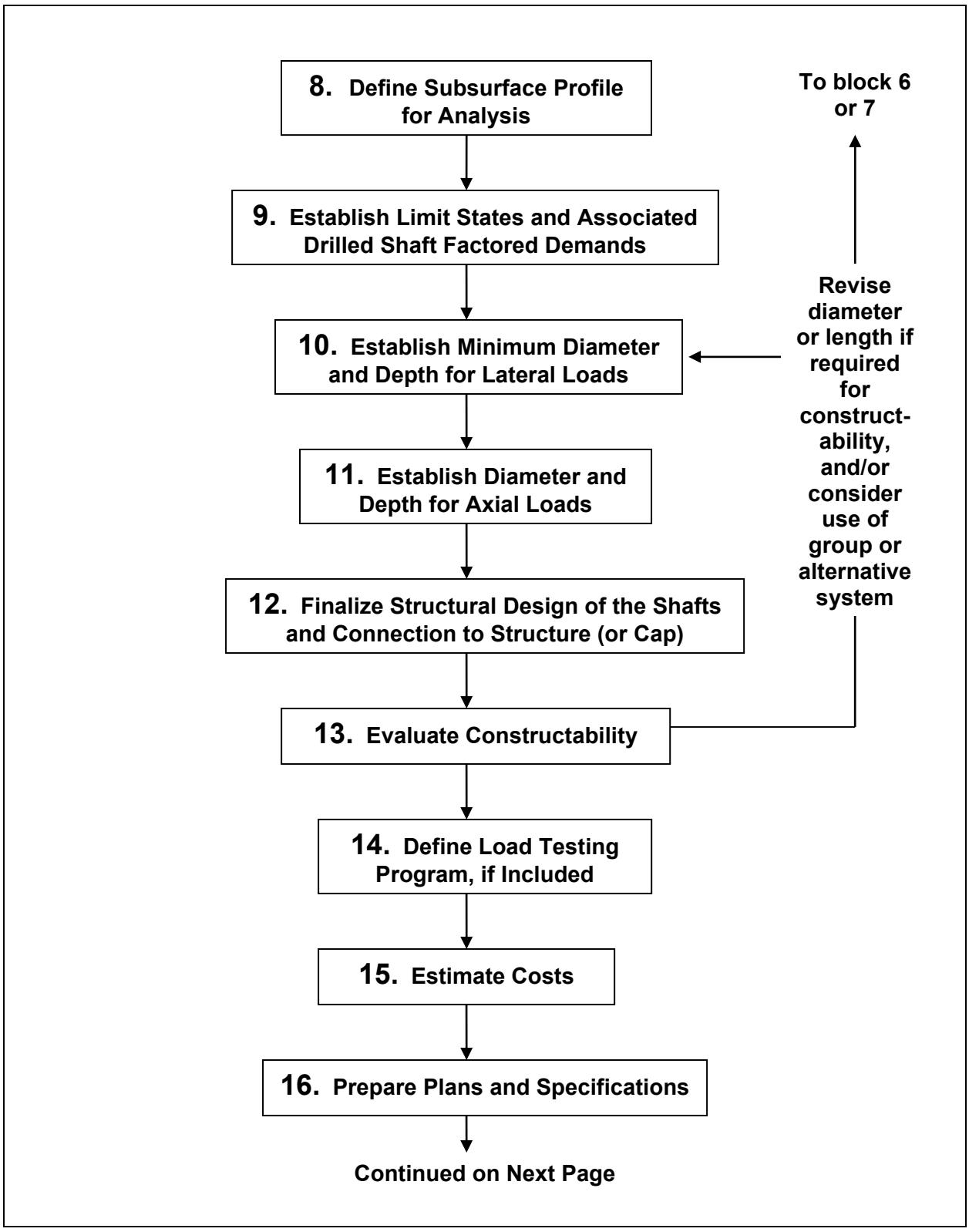


Figure 8-1 Drilled Shaft Design and Construction Process (continued)

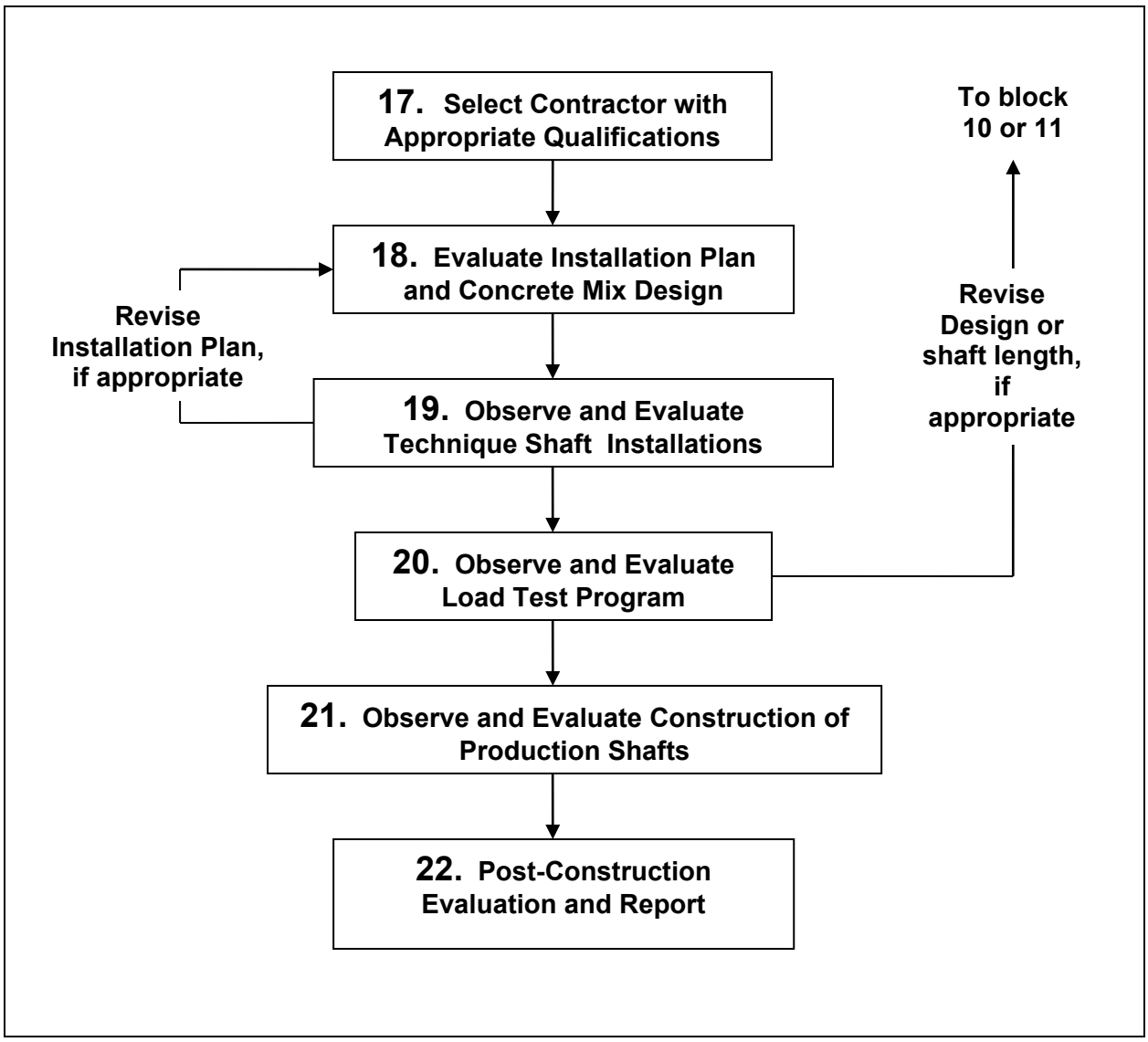


Figure 8-1 Drilled Shaft Design and Construction Process (continued)

Block 1: Establish Global Project Performance Requirements and Constraints

The first step in the entire process is to determine the general structure requirements.

1. Is the project a new bridge, a replacement bridge, a bridge renovation, a retaining wall or slope stabilization, a noise wall, a sign or light standard?
2. Are there any unusual project constraints or limitations with respect to foundation construction? Examples of constraints include accelerated work schedule, maintenance of traffic during construction, constrained workplace for construction equipment near existing structures, constrained overhead clearance, constraints from existing or abandoned foundations, existing foundations to be protected, tidal or river fluctuations, marine traffic, etc.
3. What is the most logical sequence of construction for the project; will the project be constructed in phases or all at one time? If in phases, are there potential conflicts with adjacent separate construction contracts? For example, it may be prudent to construct phase 2 foundations during phase 1 if the phase 2 construction would be very near the phase 1 structure. In some cases it may be prudent to delay some aspects of work to a subsequent phase if obstructions can be removed that will allow foundation construction to be performed with less constraints. For a replacement structure, the impact of construction on the traveling public may be minimized by constructing new foundations prior to demolition of existing structures; although the construction costs may be higher, the value to the public may be significant.
4. What are the general structure layout and approach grades?
5. What are the surficial site characteristics and general geologic setting?
6. Is the structure subjected to any special design events such as seismic, scour, downdrag, debris loading, vessel impact, etc.? If there are special design events, the design requirements should be reviewed at this stage so that these can be factored into the site investigation.
7. Are there possible modifications in the structure that may be desirable for the site under consideration? Geotechnical design professionals should participate in the decision process during the planning phase of the project, where sometimes relatively simple modifications to the structure, such as an adjustment in foundation locations, can offer significant savings or enhancements to reliability.
8. What are the approximate foundation loads? What are the deformation or deflection limits for serviceability (total and differential settlement, lateral deformations)? Note that serviceability requirements may be affected by nearby structures.
9. Are there site environmental considerations that must be considered in the design (limitations on noise, vibrations, control of drilling fluids, possible contaminated spoils)?

Block 2: Define Preliminary Project Geotechnical Site Conditions

A “desk study” combined with a site visit can often provide a substantial amount of information about the general geotechnical conditions at the site. Available information includes geologic and topographic maps, borings from previous projects at or near the site, and general knowledge within the local geology and area. A visit to a nearby quarry or road cuts can provide valuable information about the character of a potential bearing formation.

Frequently, there is information available on foundations that have been constructed in the area on other transportation or private projects, and this foundation construction experience can be quite valuable in

assessing constructability and cost-effectiveness of drilled shafts. Deep foundation trade associations are usually willing and eager to share experiences from nearby or similar projects.

Block 3: Determine Substructure Loads and Load Combinations at Foundation Level

Substructure loads and serviceability (displacement) criteria should be established for each of the load cases as described in Chapter 10. Approximate loads were considered in Block 1, and these may be only rough estimates at the time of conceptual design. Often, a set of preliminary foundation loads are developed during the preliminary design phase prior to the execution of the subsurface exploration (Block 4). It is essential that the foundation designer obtain completely defined and unambiguous set of loads and performance criteria prior to completion of the foundation design process.

Block 4: Develop and Execute Subsurface Exploration and Laboratory Testing Program for Feasible Foundation Systems

Based on the information obtained in Blocks 1 through 3, it is possible to make decisions regarding the necessary information that must be obtained for the feasible foundation systems at the site. The subsurface exploration and laboratory test program must provide sufficient and suitable information for both design *and* construction of drilled shafts and other candidate foundation systems, as described in Chapter 2. For large projects, the exploration and testing may be conducted in phases, with a preliminary exploration to define the general characteristics followed by a more detailed exploration to obtain specific design information at each foundation location.

It should also be noted that the development of the subsurface exploration program is normally preceded by a preliminary foundation design based on available information from Block 2 or from a preliminary phase exploration program (Block 4). Thus, it is expected that a rough, preliminary evaluation and design (Blocks 5 through 15) may be performed before the final exploration program is conducted. The depth and extent of borings or soundings, and the scope of the laboratory testing program, must be sufficient for the design depth and extent of anticipated deep foundation elements.

Block 5: Evaluate Information and Determine Foundation Systems for Further Evaluation

The information obtained in Blocks 1 through 4 must be evaluated to identify and select candidate foundation systems. If settlement, scour, liquefaction, footing size, etc. do not preclude the use of shallow foundations, then shallow foundations are likely to provide the most economical solution. Ground improvement techniques in conjunction with shallow foundations may also be evaluated. Shallow and deep foundation interaction with approach embankments must also be evaluated. The design of shallow foundations and ground improvement techniques are not covered in this manual. Information on design considerations for shallow foundations can be found in FHWA GEC 6 (Kimmerling, 2002). Information on ground improvement techniques can be found in FHWA GEC 13 (Schaefer *et al*., 2017).

Block 6: Deep Foundations

Where deep foundations are required, a decision must be made between drilled shafts and other deep foundation systems such as driven piles (see FHWA GEC 12, Hannigan *et al*., 2016), micropiles (Sabatini *et al*., 2005), and continuous flight auger piles (see FHWA GEC 8, Brown *et al*., 2007). Since this manual is concerned with drilled shaft foundations, alternative deep foundation systems will not be described or discussed at length. However, the relative advantages and limitations of drilled shafts are

typically considered in the light of alternative systems, as discussed in Chapter 1 of this manual. Some of the criteria considered in selection of the deep foundation system for a project include:

1. Cost. All other items being equal, the lowest cost alternative should be selected in order that the resources (tax $$) of the project owner (the taxpaying public) are utilized most efficiently. It is important that the total overall foundation cost be considered when comparing alternatives, including the pilecap, cofferdam and seal footing. For long-span bridges, a cost optimization process should consider the cost of both the foundations and the superstructure for variable span lengths.
2. Schedule. For some projects, great importance may be placed on completing the project quickly such that any schedule advantage of one type of deep foundation system may take precedence over the lowest cost alternative. Schedule impacts related to deep foundation construction may result in overall cost impacts to the project separately from the base cost of the foundation itself.
3. Constructability. Constructability and the risk of potential construction difficulties must be considered with each deep foundation alternative. Risks might also include construction impacts to nearby structures, impacts due to difficult ground conditions, undefined contaminated soil or groundwater, etc.

In general, designers should identify more than one deep foundation alternate for consideration, at least through a preliminary design.

Block 7: Select Drilled Shaft Foundations for Further Evaluation

At this point in the process, the discussion will be limited to drilled shaft foundations as the subject of this manual, although alternative deep foundation systems should normally be considered. For routine bridge projects, a common approach is to utilize a single drilled shaft to support each column. The most efficient shaft diameter and length are then determined following the process outlined in subsequent Blocks 8 through 15. As an alternative to single shaft supports, it may be more cost effective in some conditions to consider the use of groups of smaller diameter shafts with a cap. For earth retaining structures utilizing drilled shafts, it may be possible to consider secant vs. tangent piles, or some combination of drilled shafts with anchors or drilled shafts below grade with a structural wall system above. For structures extending long distances across variable conditions (such as retaining walls, sound walls, multiple light standards, etc.), it may be prudent to define separate sections of the project and consider different systems for different sections. If several deep foundation alternatives are considered, it is advisable to perform preliminary designs through Block 15 in order to evaluate the selection criteria described briefly in Block 6.

Block 8: Define Subsurface Profile for Analysis

Based on the results of the subsurface investigation (Block 4), a design subsurface profile with specific geomaterial properties must be established at each foundation location. In some cases it may be possible to group similar portions of the site for design purposes. For each design profile, it is also important that a potential range of geomaterial properties be identified so that the sensitivity of the design to each critical parameter can be evaluated during the design process. This evaluation for a range of conditions helps provide designers with the information needed to develop judgment and produce designs which are robust. Note also that different subsurface profiles may be required for different design load cases for extreme event conditions where full or partial scour may be considered or where seismic loads induce liquefaction in some layers.

Block 9: Establish the Limit States and Associated Drilled Shaft Factored Demands

As implemented in the AASHTO LRFD Bridge Design Specifications (2017a), each structural component of a bridge or other structures is required to satisfy the following equation:

$$\sum \eta_i \gamma_i Q_i \leq \sum \varphi_i R_i \qquad 8\text{-}1$$

where: η_i = a load modifier to account for ductility, redundancy, and operational importance of the bridge or other structure (dimensionless)
γ_I = load factor; a multiplier applied to force effects
Q_i = force effect
φ_i = resistance factor for resistance component *i*
R_i = nominal value of resistance component *i*

A force effect, or load demand, is defined as an axial load, shear, or moment transferred to the foundation as a result of loads acting on the structure. Article 3.4 of the AASHTO code identifies thirteen potential limit states that may require evaluation for design of a bridge. As summarized in Table 8-1, these include five limit states pertaining to strength, two pertaining to extreme events, four pertaining to serviceability, and two pertaining to fatigue. A unique combination of loads is specified for each limit state. A general description of each load combination is given in Table 8-1 and the specific loads included in each category along with applicable load factors are presented in Table 8-2. Some of the load designations shown in Table 8-2 consist of multiple load components (specifically, permanent load and live load), each of which is evaluated separately. Individual load factors assigned to the various components of permanent load are presented in Table 8-3. Additional load factor values for γ_{EQ}, γ_{TG}, and γ_{SE} are specified in AASHTO (2017a).

The most common limit states for which drilled shaft foundations are designed include: Strength I, Strength IV, Extreme Event I (earthquake), and Service I. Strength IV applies to large ratios of dead load to live load, which often governs the design of drilled shafts used to support long-span bridges over water. Any of the Strength or Extreme Event Limit States could be critical for a particular structure and could govern the foundation design. Service Limit States II, III, and IV and the Fatigue Limit States are used to check the behavior of certain superstructure elements and are not relevant to foundation design.

The limit states to be evaluated for a particular bridge or other structure are determined by the structural designer. For each limit state, the process of establishing foundation force effects can be summarized as follows. A structural model of the proposed bridge is developed and analyzed under the load combination corresponding to the limit state being evaluated (Table 8-2). Loads used in the analysis are factored. Foundation supports are modeled (typically) as springs or by using an assumed "depth of fixity." The depth of fixity models the column as being fixed at a depth that will result in the same lateral deflection as would occur in the actual column supported by the foundation. Determination of the spring constants or depth of fixity may be based on preliminary analyses of load-deformation response of trial foundation designs by the geotechnical engineer. As illustrated in Figure 8-2, reactions at the supports computed by the structural analysis are taken as the force effects (load demand) transmitted to the foundations. For drilled shafts, the reactions are resolved into vertical, horizontal, and moment components, and these are taken as the factored values of axial, lateral, and moment demand, respectively. Multiple iterations are often performed in order to obtain agreement between deformations and forces at the structure/foundation interface as calculated by the structural analysis and those based on geotechnical analysis. The resulting factored demands are substituted into Equation 8-1 (left-hand side). Although this is a somewhat oversimplified description of the actual process, it is the general procedure by which factored foundation load demands are determined, for each applicable limit state.

Note that most of the load factors given in Table 8-2 and Table 8-3 are specified over a range of values. For foundation design, modeling of the structure while varying the load factors over the specified range is necessary to determine the combination resulting in maximum force demands on the foundations, which are then used in limit state checks.

TABLE 8-1 AASHTO (2017) LIMIT STATES FOR BRIDGE DESIGN

Limit State Type	Case	Load Combination
Strength	I	Normal vehicular use of the bridge without wind
	II	Use of the bridge by Owner-specified special vehicles, evaluation permit vehicles, or both, without wind
	III	Bridge exposed to the design wind speed at the location of the bridge
	IV	Load combination emphasizing dead load force effects in bridge superstructures
	V	Normal vehicular use of the bridge with wind of 80 mph velocity
Extreme Event	I	Load combination including earthquake. The load factor for live load γ_{EQ}, shall be determined on a project-specific basis
	II	Ice load, collision by vessels and vehicles, check floods, and certain hydraulic events with a reduced live load other than that which is part of the vehicular collision load, *CT.* The case of check flood shall not be combined with *BL, CV, CT,* or *IC*
Service	I	Normal operational use of the bridge with a 70 mph wind and all loads taken at their nominal values. Also related to deflection control in buried metal structures, tunnel liner plate, and thermoplastic pipe, to control crack width in reinforced concrete structures, and for transverse analysis relating to tension in concrete segmental girders. This load combination should also be used for the investigation of slope stability
	II	Intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live load. For structures with unique truck loading conditions, such as access roads to ports or industrial sites which might lead to a disproportionate number of permit loads, a site specific increase in the load factor should be considered
	III	Longitudinal analysis relating to tension in prestressed concrete superstructures with the objective of crack control and to principal tension in the webs of segmental concrete girders
	IV	Tension in prestressed concrete columns with the objective of crack control
Fatigue	I	Fatigue and fracture load combination related to infinite load-induced fatigue life
	II	Fatigue and fracture load combination related to finite load-induced fatigue life

TABLE 8-2 LOAD COMBINATIONS AND LOAD FACTORS
(AASHTO 2017, TABLE 3.4.1-1)

Load Combination Limit State	*DC DD DW EH EV ES EL PS CR SH*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use one of these at a time				
										EQ	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-	-
Strength II	γ_p	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-	-
Strength III	γ_p	-	1.00	1.00	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-	-
Strength IV	γ_p	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-	-
Strength V	γ_p	1.35	1.00	1.00	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-	-
Extreme Event I	1.00	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-	-	-	-
Extreme Event II	1.00	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-	-
Service II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-	-	-
Service III	1.00	γ_{LL}	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-	-
Service IV	1.00	-	1.00	1.00	-	1.00	1.00/1.20	-	1.00	-	-	-	-	-
Fatigue I	-	1.75	-	-	-	-	-	-	-	-	-	-	-	-
Fatigue II	-	0.80	-	---	-	-	-	-	-	-	-	-	-	-

TABLE 8-3 LOAD FACTORS FOR PERMANENT LOADS
(AASHTO 2017, TABLE 3.4.1-2)

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.40	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (2010) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for Anchored Walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures			
◦ Metal Box Culverts, Structural Plate Culverts with Deep Corrugations, and Fiberglass Culverts		1.50	0.90
◦ Thermoplastic Culverts		1.30	0.90
◦ All others		1.95	0.90
ES: Earth Surcharge		1.50	0.75

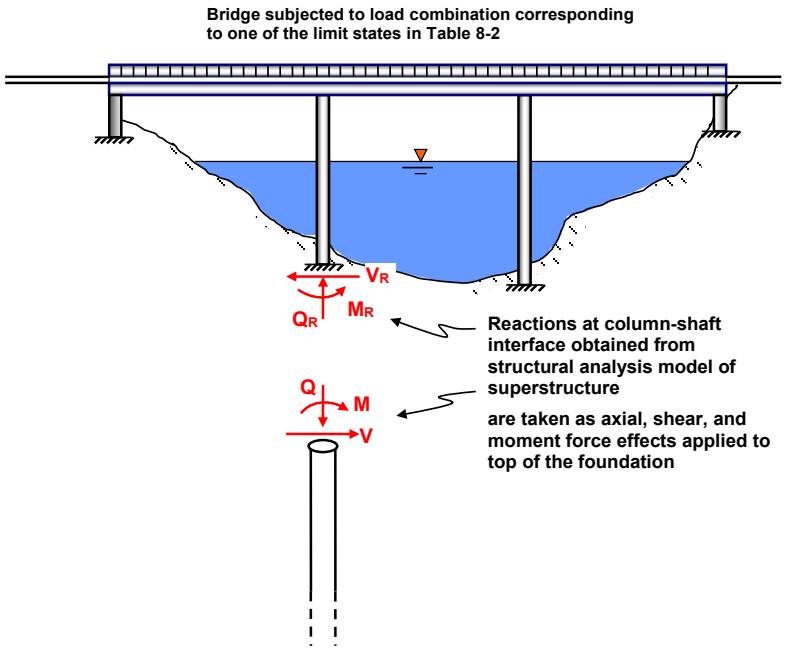


Figure 8-2 Structural Analysis of Bridge Used to Establish Load Demands at the Foundation Level

Block 10: Establish Minimum Diameter and Depth for Lateral Loads

This step is described in detail in Chapter 9, which includes a step by step flow chart of the components included in completing the design for lateral loading. Group effects for multiple shafts in a single foundation are presented in Chapter 11. In general, lateral load considerations for drilled shafts will determine a minimum shaft diameter, and so this step normally precedes the detailed design for axial loading in Block 11. In some designs, notably earth retaining structures, signs and high mast lighting, sound walls, and some bridges, lateral and overturning requirements may control the embedded length of the shaft. In such cases, it is important to evaluate the sensitivity of shaft design to geomaterial properties and the individual components of resistance to lateral loading. If design for lateral loads proves to be the controlling factor in determination of shaft tip elevation, designers should evaluate the potential benefits and costs of lateral load testing.

Block 11: Establish Diameter and Depth for Axial Loads

This step is described in detail in Chapter 10, which includes a step by step flow chart of the components included in computing axial resistance and completing the design for axial loading. Where groups of shafts are used as a single foundation (Chapter 11) subject to overturning loads, the axial load demand for individual shafts are affected by the geometric layout of the shaft group. For typical bridge structures, the design for axial loading will determine the embedded length requirement and shaft tip elevation. Note also that the use of load testing should be considered to correlate axial resistance to geomaterial properties on a site-specific basis. An evaluation of the sensitivity of the design tip elevation to individual components of axial resistance (and resistance factors, which may be affected by availability of site-specific load test information) should be performed to evaluate the cost to benefit ratio of load testing.

Block 12: Finalize Structural Design of the Shafts and Connection to Structure (or Cap)

This step is described in detail in Chapter 12, which includes a step by step flow chart of the components included in completing the structural design of the drilled shaft and the connection of the drilled shaft to the column or cap. Note that some preliminary consideration of structural design (amount of longitudinal reinforcement) is typically included in Block 10, but the final detailed design of the reinforced concrete section is completed here in Block 12.

Steps 10, 11 and 12 constitute the engineering design, in terms of establishing the dimensions (depth and diameter) and structural characteristics of a drilled shaft to meet the AASHTO LRFD criteria for all applicable limit states. To perform the calculations needed to check the LRFD criterion (Equation 8-1), resistance factors for each component of geotechnical and structural resistance must be assigned. Table 8-4 presents the resistance factors for drilled shaft design. The factors presented in Table 8-4 are consistent with those given by AASHTO (2017a) with the exception of the geotechnical lateral resistance. In that case the AASHTO code specifies a resistance factor of 1.0, whereas this manual recommends resistance factors ranging from 0.67 or 0.80 depending on the limit state under consideration. The resistance factors recommended herein are intended to be applied in conjunction with the static pushover analysis described in Chapter 9.

Block 13: Evaluate Constructability

This task should include development of a hypothetical step-by-step installation plan using the methods described in Chapters 3 through 7 of this manual. Elements of risk and/or likely construction difficulties should be identified. During this process, designers should ask themselves, "how can the drilled shaft design be modified to minimize risk of construction difficulties while still meeting performance requirements and allowing contractor flexibility in selecting their means and methods of drilled shaft construction?" This exercise may also identify potential construction practices or sequence of work which could adversely affect performance for this specific project so that any such practices can be specifically excluded in the project special provisions. If modifications to the design are indicated or potential improvements identified, recycle through Blocks 10 to 12 or 6 to 12, as needed. An independent constructability review by an experienced person can be very valuable in this effort. The ADSC: The International Association of Foundation Drilling is a trade association that offers to provide constructability review by their subcontractor members through regional chapters located across the U.S.

Block 14: Define Load Testing Program, if Included

Load testing can provide valuable information for site specific evaluation or verification of the design, and assessment of any influence of the contractor's selected means and methods of construction on the performance of the drilled shafts. The use of load testing is a factor in the choice of resistance factors. Consideration of the potential benefits and costs of load testing should be included in Blocks 10 and 11. If load testing is to be included in the project, designers must define a potential location on the site which is representative of typical subsurface conditions for the project as defined in Block 8. Details of the specific load testing program must address the specific components of resistance which are most important to the design, as identified by sensitivity studies in Blocks 10 and 11. Load testing is discussed in detail in Chapter 13 of this manual.

TABLE 8-4 RESISTANCE FACTORS FOR LRFD DESIGN OF DRILLED SHAFTS

Limit State	Component of Resistance	Geomaterial	Equation, Method, or Chapter Reference	Resistance Factor, φ
Strength I through Strength V Geotechnical Lateral Resistance	Overturning of individual elastic shaft; head free to rotate	All geomaterials	*p-y* method pushover analysis; Ch. 9	0.67
	Overturning of single row, retaining wall or abutment; head free to rotate	All geomaterials	*p-y* pushover analysis	0.67
	Pushover of elastic shaft within multiple-row group, w/ moment connection to cap	All geomaterials	*p-y* pushover analysis	0.80
Strength I through Strength V Geotechnical Axial Resistance	Side resistance in compression/uplift	Cohesionless soil	Beta method	0.55 / 0.45
		Cohesive soil	Alpha method	0.45 / 0.35
		Rock	Eq. 10-21	0.50 / 0.40
		Cohesive IGM	Modified alpha method	0.60 / 0.50
	Base resistance in compression	Cohesionless soil	N-value (Eq. 10-14)	0.50
		Cohesive soil	Bearing capacity eq.	0.40
		Rock	Eq. 10-23 or Eq. 10-29	0.50
	Static compressive resistance from load tests	All geomaterials		0.70
	Static uplift resistance from load tests	All geomaterials		0.60
	Group block failure	Cohesive soil		0.55
	Group uplift resistance	Cohesive and cohesionless soil		0.45
Strength I through Strength V; Structural Resistance of reinforced concrete	Axial compression			0.75
	Combined axial and flexure			0.75 to 0.90
	Shear			0.90
Service I	All cases, all geomaterials		Chapters 9, 10	1.00
Extreme Event I and II	Axial geotechnical uplift resistance	All geomaterials	Methods cited above for Strength Limit States	0.80
	Geotechnical lateral resistance	All geomaterials	*p-y* method pushover analysis; Chapter 9	0.80
	All other cases	All geomaterials	Methods cited above for Strength Limit States	1.00

Block 15: Estimate Costs

A cost estimate should be prepared for the foundation as designed (including any alternate designs) with consideration of any special constructability issues identified in Block 13 that may affect costs. In most cases, this block should be included as a part of the project planning stage and during preliminary design when several alternates may be considered. During the final design process, this block will serve as input to the engineer's estimate of project cost prior to bid. Cost estimation is discussed in detail in Chapter 18 of this manual.

Block 16: Prepare Plans and Specifications

When the design has been finalized, plans and specifications can be prepared and the procedures that will be used to inspect and establish final tip elevation in the field can be defined. It is important that all of the quality control procedures are clearly defined for the bidders to avoid claims after construction is underway. If there are to be separate pay items for different components of work, these items must be defined in an unambiguous way in order to avoid disputes. Examples include payment for obstructions, earth excavation vs. rock excavation, permanent casing, etc. If rock excavation is anticipated and is not defined as a separate pay item, then the amount of rock to be removed should be defined to the extent possible, including depth of rock sockets as well as the decomposed or fractured rock that may be removed but not considered part of the socket for design purposes. Note also that construction of drilled shafts is specialty construction work which requires a contractor or subcontractor with the appropriate equipment, skill, and experience. Requirements for qualification or prequalification of foundation contractors should be included in the bid documents. Requirements for contractor submittals and timelines for review of submittals must also be included in the contract documents. Specifications, inspection, and tests for completed shafts are described in detail in Chapters 14 through 17 of this manual.

Block 17: Select Contractor with Appropriate Qualifications

After the bidding process is complete, a successful contractor meeting the qualifications defined in Block 16 is selected in accordance with the selection criteria established by the contracting agency.

Block 18: Evaluate Installation Plan and Concrete Mix Design

The review of the contractor's installation plan by the designer (and often by an independent reviewer for large projects) is a critical component to successful construction of drilled shafts that are consistent with the design requirements. Constructability concerns that may have been identified in Block 13 should be addressed, and the inspection and quality control procedures (Block 16) associated with the proposed installation method should be defined. The installation plan should always be checked for consistency with the project plans and specifications. The submittal of the proposed concrete mix, and related concrete test data, must be reviewed to assess the suitability of the concrete mix to the specific requirements and constraints of the project, as discussed in Chapter 7. A preconstruction meeting is advised so that the designer, inspector, and contractor can review and discuss the most important issues.

Block 19: Observe and Evaluate Technique Shaft Installation

At least one technique shaft installation should be performed as part of the project requirements prior to installation of load test shafts and prior to the start of production shaft construction to demonstrate that the installation plan described in Block 18 is suitable for the project. If adjustments to the installation plan are required, additional review by the project designers is necessary. A separate technique shaft should be installed for each of the different installation methods proposed by the contractor, and for each of the subsurface site conditions defined within the project. For small projects, consideration can be given to combining the technique shafts with the load test shafts noted in Block 20.

Block 20: Observe and Evaluate Load Test Program

If a load test program is included during the construction, the installation of the test shaft should normally occur after successful completion of a technique shaft installation in Block 19. It is important that the load test shaft be constructed in a manner and in a location that is representative of the production shafts, and inspection of the construction of this shaft must carefully note the actual conditions encountered. The results of the load test should be evaluated by the designers for the purpose of correlating the measured resistance with geomaterial properties used in the design. Adjustments to the final design and anticipated tip elevations may be made if justified by the load test data. It is important that the final drilled shaft installation criteria developed from the load test program should include allowance for anticipated variations in stratigraphy and geomaterial properties across that site and that the load test results be interpreted with due consideration of the likely variations between foundation locations. Field load testing is discussed in Chapter 13 of this manual.

Block 21: Observe and Evaluate Construction of Production Shafts

The procedures for observation, inspection, and documentation of the installation of production shafts are defined in Block 16, modified if necessary in Blocks 18 through 20, and implemented in this Block. An important component of the inspection process is the communication between inspectors, designers, and contractors so that any unusual condition or problem can be addressed in a timely and efficient manner. Post-installation integrity testing is a basic element of all drilled shaft construction to verify that the drilled shafts were successfully installed without the presence of significant defects that can adversely affect the required structural or geotechnical performance of the foundation. If defects are noted, the designers will need to assess the potential impact on the performance requirements, evaluate the need for remediation, and evaluate the effectiveness of any potential remediation of defects. Inspection and construction records for drilled shafts are discussed in Chapter 15 of this manual. Techniques for investigating the structural integrity of the completed drilled shaft installation are discussed in Chapter 16, and strategies for assessment and repair of defects are described in Chapter 17.

Block 22: Post-Construction Evaluation and Report

A final report should be prepared to document the successful construction of each drilled shaft installation, and to document the final as-built conditions of the foundation. Where remediation measures are performed to correct deficiencies in a drilled shaft, the corrective measures and subsequent verification test data should also be documented in the final report and as-built records. This documentation is important for future projects when rehabilitation or modification or replacement of the structure may be required. Using these records, well-constructed and documented foundations can often be incorporated into a new structure at the same location. In addition, every project offers opportunities for lessons learned; documentation of the drilled shaft construction and testing can be a valuable reference to the owner and to designers for future projects in similar circumstances.

8.2 CALIBRATION OF RESISTANCE FACTORS TO REGIONAL CONDITIONS OR AGENCY PRACTICE

The resistance factors presented in this manual as well as in AASHTO (2017a) are intended to cover a wide range of conditions commonly encountered by transportation agencies involved in drilled shaft design using LRFD. However, given the wide range of geologic environments, natural variability of geomaterials, and different construction practices, there will always be design problems that do not fit within the general framework of these methods. Moreover, design methods with carefully calibrated resistance factors that are specific to local or regional geologic conditions and construction practices offer the potential for cost-effective and safe designs that work well for the agency willing to invest in their development.

A common starting point for converting existing ASD design methods to LRFD format is to use fitting to the factor of safety used in current practice. It is emphasized that calibration by fitting does not address the variability or bias of the prediction method, and it is not possible to assess the probability of failure. Whatever margin of safety was implied by the ASD safety factor, it is simply carried over to the LRFD format without any change. Fitting should therefore be considered an interim approach, or as a check on reliability-based calibrations when such calibrations are available.

Allen et al. (2005) provides a detailed description of the process used to perform calibration of load and resistance factors as applied to limit state conditions for LRFD structural and geotechnical design, including the information needed for such calibrations. This publication is a highly recommended resource for agencies wishing to develop in-house or regionally specific design methods for drilled shafts. For each limit state of interest, the general steps include:

- Compile data on loads (force effects) and resistances; data on resistances is obtained from field load tests on drilled shafts
- Calculate the statistical parameters for characterizing loads and resistances (mean, standard deviation, type of distribution, bias)
- Establish a target level of reliability
- Perform the calculations from reliability theory to determine resistance factors

As a final note, it is important to recognize that the application of design equations using LRFD, or any method, is only one component of ensuring structural safety of drilled shafts. Other sources of risk are associated with site investigation, construction, inspection, and competent engineering throughout the project. No design equations will ensure safety if soil properties used in those equations are incorrect or if the drilled shaft is not constructed with proper control of quality.

8.3 SUMMARY

This chapter provides a roadmap of the overall design process for drilled shaft foundations, with a step by step description of the tasks to be completed. The steps may not always be performed in the linear fashion shown, and most of the time there will be several iterations of many of the individual steps through the process of planning and conceptual design, preliminary design, and final design. The precise order of the steps in the flow chart is not so important; rather, it is important that all of the steps in this checklist be completed. In subsequent chapters of this manual, many of the blocks shown in this chapter are further subdivided into more detailed flow charts of the individual tasks.

CHAPTER 9
DESIGN FOR LATERAL LOADING

9.1 INTRODUCTION

Design for lateral loading typically controls the diameter of drilled shafts for highway bridges and may also control the embedded length for some types of structures such as retaining walls, noise walls, and sign or light standard foundations. Thus, an evaluation of lateral loading is required during planning and preliminary design. A more complete analysis of lateral loading conditions is required for final design including both geotechnical and structural design; structural design of drilled shafts is covered in Chapter 12 of this manual. This chapter on design for lateral loading provides examples of applications in highway structures, presents significant concepts, and will address the details of standard and routine design of drilled shafts for lateral loading. A more rigorous coverage of laterally loaded deep foundations can be found in the FHWA Geotechnical Engineering Circular 9 (GEC-9) publication on "Design, Analysis and Testing of Laterally Loaded Deep Foundations," by Parkes, et al. (2018).

This manual does not explicitly address the design of drilled shaft foundations for seismic loading; however, many of the principles described in this chapter can be used for the determination of linear or nonlinear constraints (spring constants) for the bases of structural columns for the purpose of performing dynamic analysis of the structure.

9.2 EXAMPLES OF LATERAL LOADING APPLICATIONS

The principal use of drilled shafts in highway projects is for supporting bridge piers and abutments, but they can also be used in the construction of retaining walls, overhead signs, sound walls and for slope retention. The lateral loads that are exerted on drilled shafts for highway structures are derived from earth pressures, centrifugal forces from moving vehicles, braking forces, wind loads, current forces from flowing water, wave forces in some unusual instances, ice loads, vessel impact and earthquakes, which are viewed as "extreme events" with a probability of occurrence that exceeds the design life of the bridge. Even if none of the above sources of lateral loading are present, an analysis of a drilled shaft may be necessary to investigate the deformations and stresses that result within a drilled shaft from the intentional or unintentional eccentric application of axial load and from accidental batter. Examples of some cases in highway construction where drilled shafts are subjected to lateral loading are given in the following paragraphs. Analytical techniques are then described, along with examples of their use.

Monoshaft (single drilled shaft supporting an individual column) supports offer advantages in the small footprint produced by the foundation for circumstances where the geometry of the structure, site access limitations or other factors discourage multiple columns and foundation groups. For aesthetic reasons, it is becoming more popular to use single columns instead of rows of columns in bents, especially for tall structures. Retrofitting and rehabilitation work or bridge widening may also dictate the use of single column support with monoshaft foundations. Monoshaft foundations are also often a more economical type of foundation in comparison to foundations with multiple elements and a pile cap. Figure 9-1 illustrates single-column bents which use a monoshaft foundation that is continuous with the column.



Figure 9-1 Single Column Piers with Monoshaft Foundations

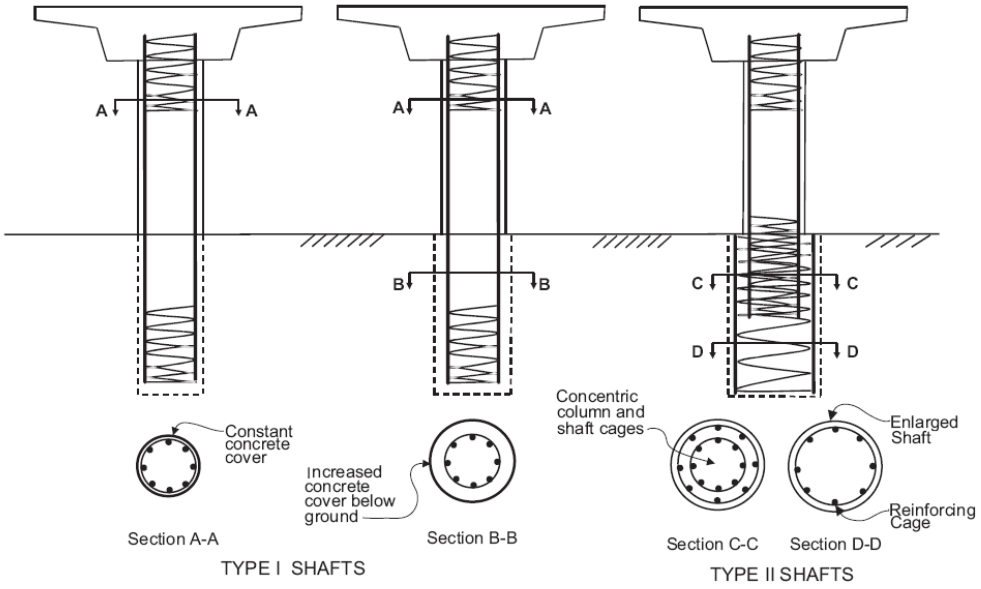


Figure 9-2 Monoshaft Foundations Used by Caltrans (Caltrans Seismic Design Criteria, Version 1.4, June, 2006)

Two approaches to the design of monoshaft foundations are typified by the "Type I" and "Type II" designs used by Caltrans, as illustrated in Figure 9-2. The Type I foundation provides a continuous reinforcing cage from the drilled shaft to column so that the foundation is essentially an extension of the column into the subsurface. The Type II foundation includes a drilled shaft with a significantly larger diameter (by at least 18 inches) and a larger reinforcing cage than used for the column. This approach is intended to ensure that, should overstress in flexure occur during a seismic event, a plastic hinge would form at the base of the column / top of the shaft rather than at depth where inspection and/or repair would be more difficult.

Where lateral (and/or vertical) loads are relatively large, groups of drilled shafts may provide a more efficient foundation solution, particularly where multiple columns may be supported on a single foundation. Groups of drilled shafts may also be needed to resist large lateral loads from vessel impact, seismic forces, ice load, or wind on high bridges that can produce large shear and overturning forces at the base of the column. If scour, liquefaction, or deep water conditions result in long unsupported shaft lengths, the lateral strength or stiffness of a monoshaft foundation may not be sufficient or may be

impractical due to the large diameter shaft required. With a group of shafts connected by a common pile cap, the cap provides rotational restraint for lateral load at the top of the shaft, and transfers column bending moments into axial forces on the shafts. The force couples resulting from the axial resistance of separate shafts provides rotational strength and stiffness. A drilled shaft group may also be more effective in controlling lateral displacement and rotation of the foundation.

The diagram shown on Figure 9-3 illustrates a group of eight shafts used to support a pair of rectangular columns in a pattern similar to that used for the replacement bridge for the I-35W structure across the Mississippi River in Minneapolis. Each of the main piers for the pair of new bridges has two rectangular columns supporting a segmental box girder. The two columns are supported on a group of eight 7-ft diameter drilled shafts. The shafts are excavated through soils subject to scour and are embedded approximately 40 ft into a sandstone bearing stratum.

The basic principles for design of individual shafts within the group are described in this chapter. The design of groups of drilled shafts for combined lateral and axial loading is described in detail in Chapter 11 of this manual.

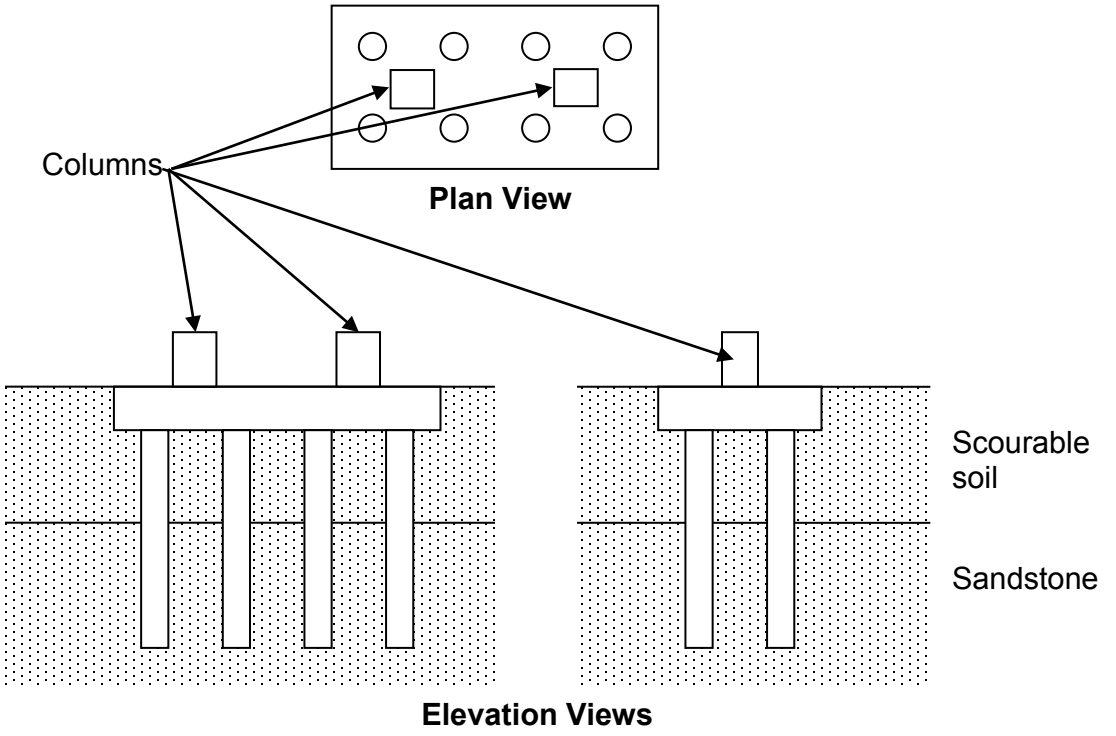


Figure 9-3 Example of a Drilled Shaft Group Foundation for a Bridge

A bridge over open water is subjected to lateral forces that include wind loads, current and wave forces, ship or barge impact, possibly seismic loads or ice loads, and centrifugal forces and braking forces resulting from traffic. Braking forces could be sizeable, especially if heavily-loaded trucks are suddenly brought to a stop on a downward-sloping span. It is also possible that the soil surface around the foundations could be lowered due to scour of soils from normal stream flow as well as from flood conditions and storm surge.

A bridge over water may often have relatively tall columns as shown on the example in Figure 9-4, where the height of the structure increases the overturning moment at the base of the column from wind, traffic, and seismic loads. Note the use of a strut between columns in Figure 9-4. A strut between columns can help engage the resistance of multiple shafts for vessel impact loads without using a foundation cap.



Figure 9-4 Drilled Shafts for Bridge Over Water, Somerset, KY

Sound walls, sign structures, and high mast lighting are examples of structures which are relatively lightly loaded in the vertical direction but subject to significant lateral shear and overturning moments due to wind. Wind loads act against the projected area of the structure, and wind gusts produce a force which is transient and often cyclic. Vehicle impact forces may also be significant design components in some instances. Figure 9-5 shows views of two types of foundations used for sign structures. Figure 9-5a shows a two-shaft foundation, and Figure 9-5b shows a single-shaft support. The two-shaft system resists the wind moment largely by added tension and compression (a "push-pull" couple) in the shafts, although some bending is required to resist the shear force, while the single-shaft foundation resists both the moment and shear produced by the wind load through bending.

Another characteristic of these types of structures is the fact that borings may be either widely spaced or infrequent so that the subsurface conditions at the location of an individual foundation element are known with less reliability than might be the case for a more substantial structure such as a bridge foundation. In such a case, the designer must consider the potential variation in subsurface conditions for which a design might be used so as to accommodate a broad range of possible circumstances.

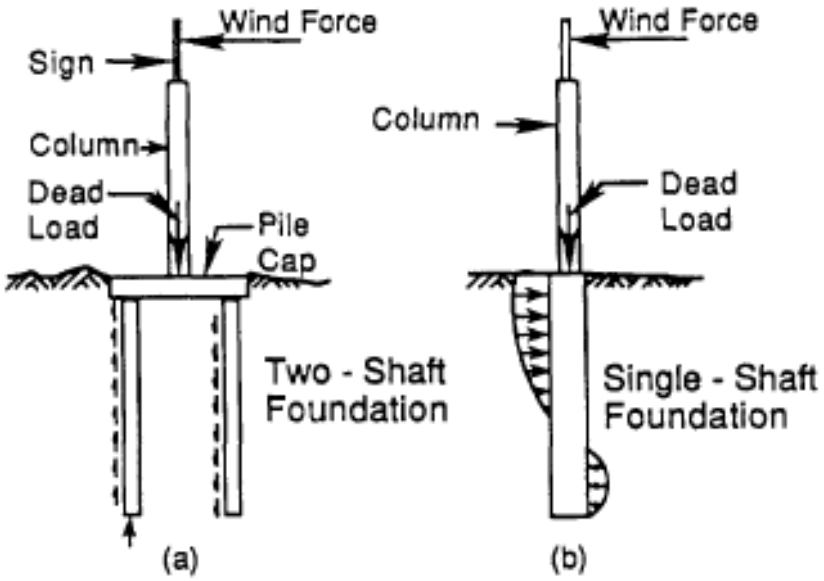


Figure 9-5 Elevation View of an Overhead Sign Structure: (a) Two-Shaft Foundation; (b) Single-Shaft Foundation (from *FHWA-IP-84-11*)

In a recent survey of hurricane wind damage to structures in Florida (Jones, 2005), the most commonly observed failure mode for sign foundations was due to structural failure in the anchor bolt connection at the top of the shaft. In at least some cases there were failures due to poor quality concrete at the top of the shaft or misalignment of the anchor bolts with the drilled shaft. The strength of this connection is typically the weak link (by design) in the system, but there is a need to ensure good workmanship in the placement of anchors and in the completion of the concrete placement.

The lateral loadings acting on abutments result from soil pressures from the backfill acting on the abutment (which can be affected by settlement and/or lateral creep), traffic surcharge loads, braking forces that are transmitted through the deck system, thermal expansion or contraction of the bridge structure, seismic forces, and possibly other sources. Drilled shafts can carry large lateral loads because they can be installed with large diameters; their flexural strength is such that the loads from abutments can often be supported without the need to batter the shaft, as is commonly done for driven piles to resist lateral loads.

Figure 9-6 illustrates three different applications of drilled shafts for bridge abutments. Figure 9-6a shows a conventional cast-in-place concrete abutment founded on two rows of drilled shafts; this type of footing resists overturning of the abutment by the compression and tension loads developed in the front and back rows of drilled shafts. Figure 9-6b illustrates a spill-through or "stub" abutment which relies on a soil slope beneath the bridge structure to provide lateral support for the drilled shafts and to prevent loss of soil from between the drilled shafts. The drilled shafts in Figure 9-6c are installed either directly against the adjoining drilled shafts ("tangent pile" wall) or slightly overlapping with the adjacent drilled shafts ("secant pile" wall). With either a spill-through abutment or a tangent/secant pile abutment, the drilled shafts resist the applied lateral loads entirely in bending.

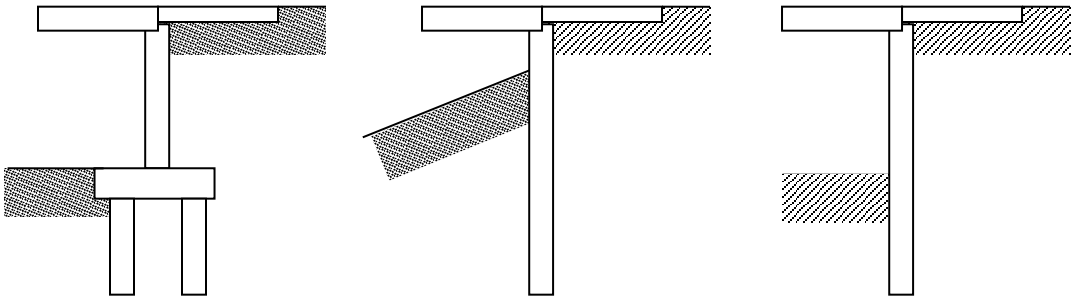


Figure 9-6 Sketch of Foundation Arrangements for a Bridge Abutment



Figure 9-7 Drilled Shaft Abutment Foundations

The photographs of example abutments shown in Figure 9-7 also illustrate the use of drilled shafts to construct abutment walls. The tangent pile wall on the left will be covered with a precast concrete curtain wall facing, and the drilled shafts for the abutment on the right will support a conventional abutment wall above grade.

Although the abutments described above are typically designed to resist lateral forces from the soil pressure behind the abutment, an arch bridge can also result in relatively large lateral thrust forces into the abutments. The arch bridge shown in Figure 9-8 includes thrust blocks designed to transmit the force from the arch into the abutment foundation, which is supported on vertical drilled shafts.

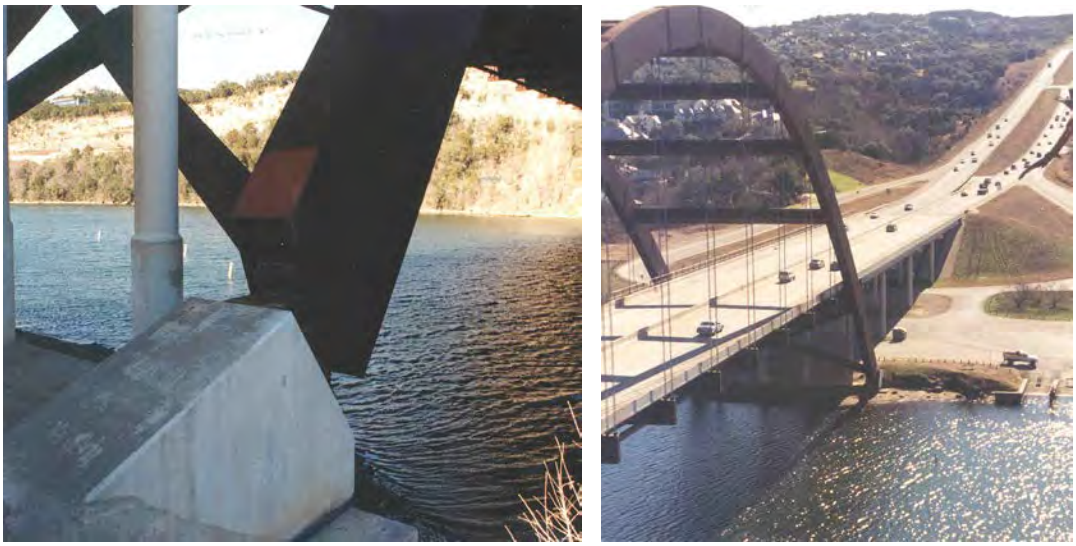


Figure 9-8 Arch Bridge (photograph courtesy of Ronald C. O'Neill)

Drilled shafts may be used to construct earth retaining structures for highways, either to form the wall itself or as foundation support for a conventional wall. Large diameter drilled shafts can be drilled vertically either with some overlap (secant pile wall) or immediately adjacent to one another (tangent wall) or with limited clear spacing between the piles. If the depth of excavation in front of the wall of drilled shafts is too large for the shafts to carry the lateral loads as cantilever elements, they can be tied back with ground anchors. Figure 9-9 illustrates a secant pile wall (Figure 9-9a) during construction, and a depressed section of highway in which the excavation has been retained by drilled shafts used as tangent piles (Figure 9-9b). Figure 9-10a provides a schematic of a retaining wall foundation on drilled shafts, in which the shafts are subject to lateral shear and overturning from the earth pressures on the precast panels of the wall above. Figure 9-10b shows such a wall under construction with the coated steel H beams extending up from the drilled shaft soldier piles; the panels have not yet been added and backfilled.



(a) (b)

Figure 9-9 Drilled Shaft Retaining Walls: (a) Secant Wall; (b) Tangent Wall



(a) (b)

Figure 9-10 Drilled Shaft Foundation for a Retaining Wall using Soldier Piles and Precast Panels

Another use of drilled shafts to resist lateral loads is in problems of slope stability, as illustrated in Figure 9-11 and Figure 9-12. Some of the forces from the moving soil mass are transferred to the upper portions of the drilled shafts, which serves to increase the resisting forces in the soil, with a resulting increase in the factor of safety. The portion of a drilled shaft below the sliding surface must be designed to resist the applied forces without excessive deflection or bending moment.

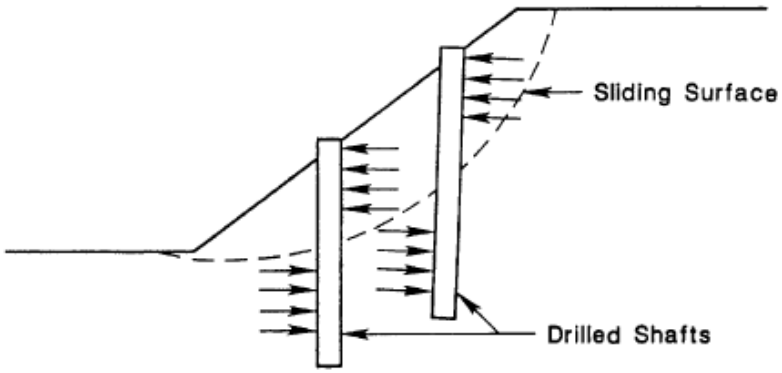


Figure 9-11 Drilled Shafts for Stabilizing a Slide (Reese et al., 1987)

Drilled shafts can provide several advantages as a means of stabilizing a slope. The construction of a drilled shaft will usually cause less soil disturbance than driving a pile. Crane-mounted drilling machines can be rigged so that the machine can sit above or below the slope and reach 80 ft or more horizontally to drill the shaft. Micropiles, anchors, and other types of deep foundations can also be used, but these more slender elements have less flexural strength and are installed so as to mobilize primarily axial resistance.

A similar problem occurs with earthquake-induced lateral spreading due to liquefaction. With liquefaction, the soil resistance in the liquefied zone is affected by the transient and elevated pore water pressures. A soil layer which undergoes liquefaction produces a weak zone which can result in large lateral soil movements at the location of a drilled shaft foundation. The foundation can be used to resist the lateral soil movements and thus stabilize the sliding mass, or the foundation may be simply subject to

passive soil pressures as a "flow-around" type of ground movement occurs. Unless a liquefaction mitigation treatment is employed, the foundation must be designed to resist the effects of this extreme-event loading.

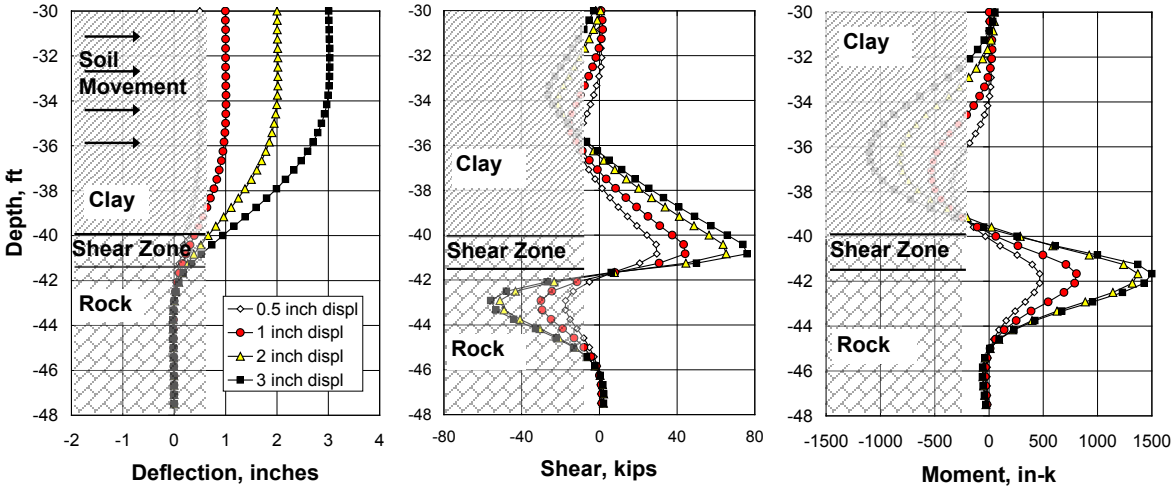


Figure 9-12 Analysis of Drilled Shafts in Moving Soil (Loehr and Brown, 2007)

9.3 DESIGN FOR LATERAL LOADING

9.3.1 Design Process

For lateral loading, the design can be controlled by geotechnical or structural strength requirements or by serviceability (deformation) conditions. These conditions are described as follows:

1. Geotechnical Strength Limit State (resistance of the shaft to overturning). The shaft should be of sufficient size and penetrate to sufficient depth to support the factored design loads without collapse. In general, deflections are not a controlling consideration for this condition; however, a computed deflection which is sufficient to cause collapse of other portions of the structure could represent a strength limit. The most critical lateral loading conditions affecting the geotechnical strength limit state are often associated with transient wind or extreme event load cases.
2. Structural Strength Limit State (strength of the shaft in flexure and shear). The shaft should be of sufficient size and constructed with the necessary reinforcement to resist the bending moment, shear and axial loads that will be imposed on the drilled shaft.
3. Service Limit State (deformations). The shaft should be of sufficient size and depth that the lateral deformations and rotation under service load conditions are within tolerable levels of the structure at the critical locations (typically at the top of the column). Design for lateral loading must include a determination of the deformations and/or stiffness of the drilled shaft in lateral translation and rotation so that the effects of foundation deformation can be considered in the analysis of the structure.

The design process for lateral loading is represented by Block 10 in the overall design process described in Chapter 8 of this manual. This block is subdivided in Figure 9-13 to illustrate the process of design for lateral load.

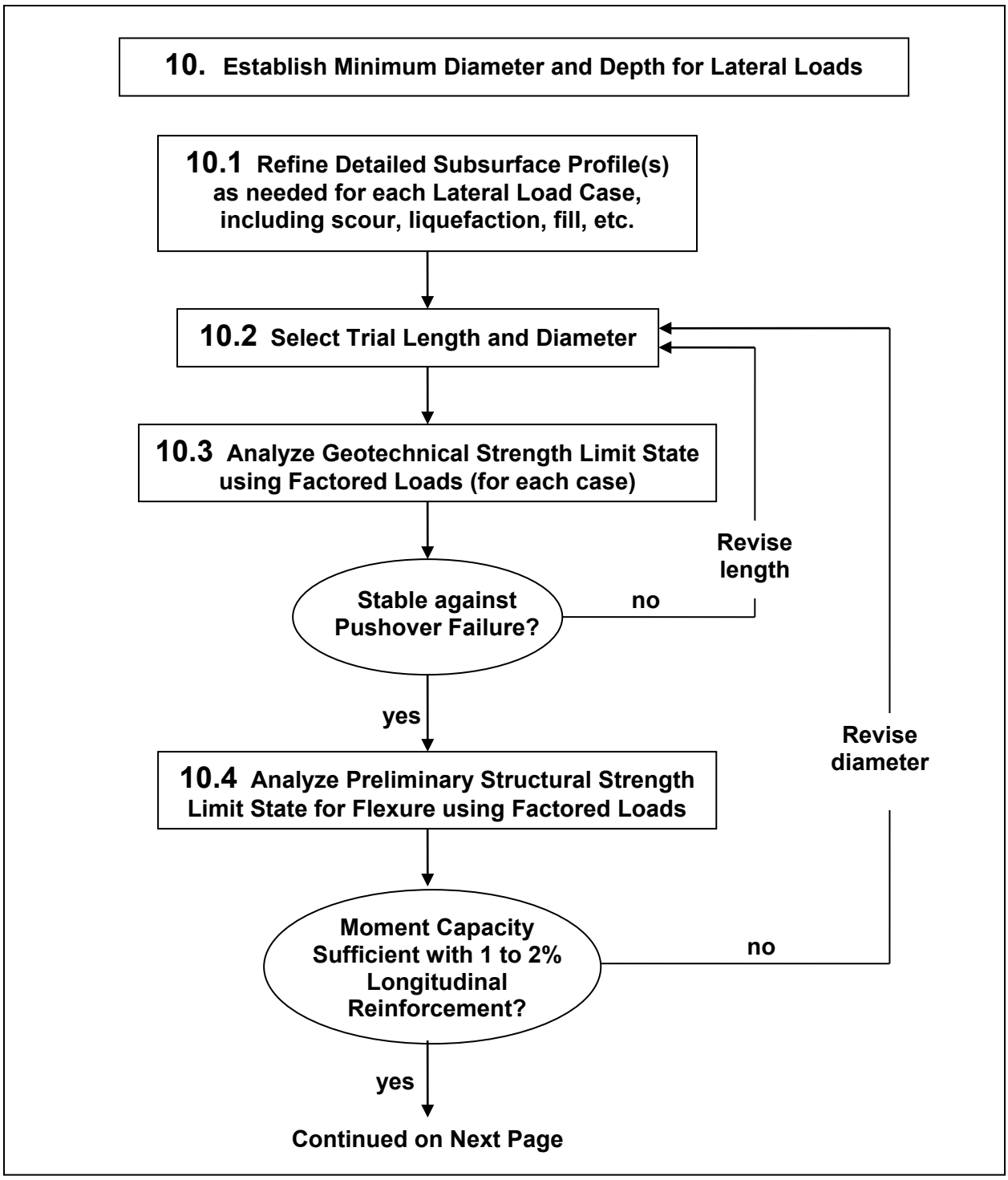


Figure 9-13 Drilled Shaft Design Process for Lateral Loads

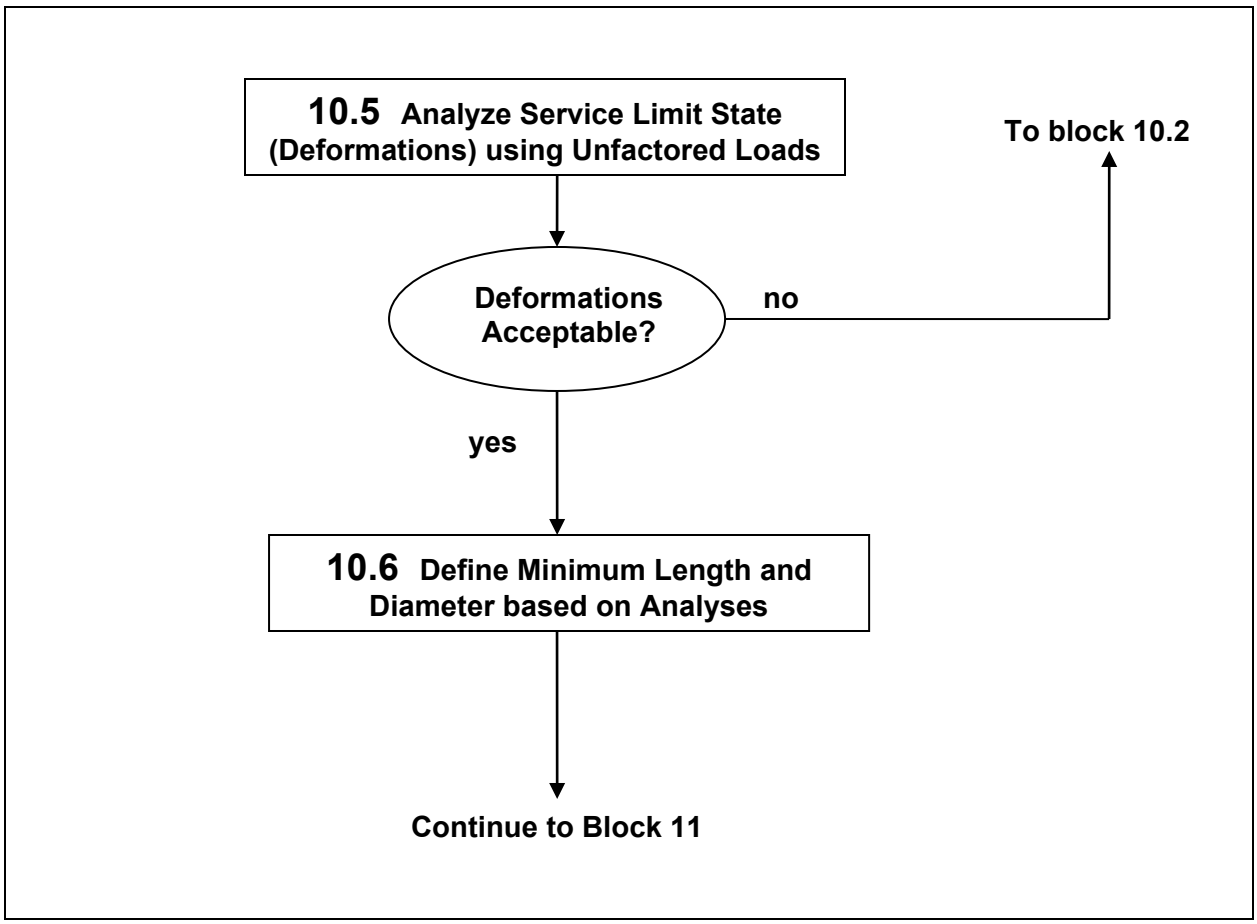


Figure 9-13 Drilled Shaft Design Process for Lateral Loads (Continued)

The information obtained in the site subsurface exploration and testing program described in Chapter 2 are used to develop a design subsurface profile for each lateral load case as required in Block 8 in Figure 8-1 of Chapter 8 and serves as the initial step (Block 10.1) for the design of the drilled shafts for lateral loads. The variability of geomaterial properties and strata elevations should be considered in developing a design profile. As part of the subsequent design process, the designer will need to evaluate the sensitivity of the lateral load resistance to the various input parameters relating to the subsurface profile. To achieve a design that accommodates possible variability, it may be necessary to evaluate more than one design subsurface profile for a given foundation location.

Besides variation in stratigraphy across the site, different load cases may require different profiles because of the effects of scour or liquefaction. Scour associated with the "design flood" affects the soil resistance for the strength limit states. Scour associated with the "check flood" is an extreme event design condition and is considered with different load and resistance factors. Lateral load considerations for the design scour and for extreme event loads and conditions are briefly described in Section 9.3.3.5.

The remainder of this chapter will focus on the design process for lateral loading for routine design, as outlined in Blocks 10.2 through 10.6. To design a drilled shaft for lateral loading, the engineer should have the capability to compute deflection and rotation at the head of the drilled shaft and the maximum bending moment and shear force in the embedded drilled shaft. The engineer should also have the capability to compute the bending moment for a reinforced-concrete section at which a plastic hinge will develop, termed the nominal bending moment capacity. The nominal bending moment capacity depends

on the magnitude of the axial force acting upon the drilled shaft. All these factors depend to a large degree on the reaction provided by the soil or rock as the drilled shaft translates laterally beneath the ground, and on the relationship between bending moment and rotation of the drilled shaft cross section. Both of these inherently nonlinear effects are addressed in this section.

9.3.2 Planning Stage Estimates

Lateral load considerations and flexural strength requirements often control the minimum shaft diameter. In cases of a single drilled shaft foundation for a single column, these strength requirements are often reflected in the column size. Preliminary shaft diameter is often set as slightly larger than that of the column. The longitudinal reinforcement may be continuous with the column (even though greater cover is typical for a drilled shaft than for a column), or the longitudinal reinforcement for the shaft may include a larger diameter cage to accommodate larger bending moments for the shaft relative to the column, such as may be needed for drilled shafts with long unsupported length at water crossings.

For single drilled shaft foundations, it is usually desirable to include a larger diameter drilled shaft even if the longitudinal reinforcement in the shaft will match that of the column. The cover on the reinforcement in the shaft can be designed to accommodate the typical 3 to 6-inch tolerance needed for the constructed location of the shaft and still allow the reinforcing to be positioned within the shaft excavation to line up with the column with the required minimum cover in the "as-built" condition.

9.3.2.1 Preliminary Estimate of Maximum Bending Moments (Demand)

To select a trial diameter and length in Block 10.2, it is helpful to perform a very simple preliminary analysis for structural strength to estimate the approximate maximum bending moment in the shaft. An estimate of factored loads at the top of the shaft is required to perform the preliminary analysis. The structural design of a typical bridge structure is conducted using some type of frame analysis, with force effects resolved at the top of shaft or near the ground line, as illustrated in Figure 9-14. Ground line forces from signs, light standards, sound walls, and similar elements are usually computed using simple statics.

For preliminary design, a simple first estimate of maximum bending moment in the shaft below grade can often be performed by modeling the shaft below grade as a free-standing column of some length, which is fixed at the base. Although there exist many misconceptions among engineers regarding the concept (often referred to as a "point of fixity"), the concept can be used effectively for preliminary design if the limitations are understood. In essence, the more realistic nonlinear model of the shaft as a beam with nonlinear p-y springs used to model the soil resistance along its length (described subsequently and illustrated in Figure 9-14a) is replaced by a simple equivalent linear system composed of a free-standing cantilever beam which is fixed at the base (Figure 9-14b). The results of analyses using a p-y model (described subsequently) suggest that the equivalent cantilever lengths presented in Figure 9-15 would provide a reasonable estimate of maximum bending moment in the shaft which is approximately equal to that computed using a more sophisticated model.

Figure 9-15 provides an equivalent "fixity" length, expressed in shaft diameters, which may be used to estimate the maximum bending moment in the shaft below grade, as outlined on Figure 9-14. Note that the maximum bending moment generally does not occur at a point below grade corresponding to L_{equiv}. This model is simply an approximation to estimate maximum bending moments in the shaft for preliminary design; there is really no such thing as a "point of fixity."

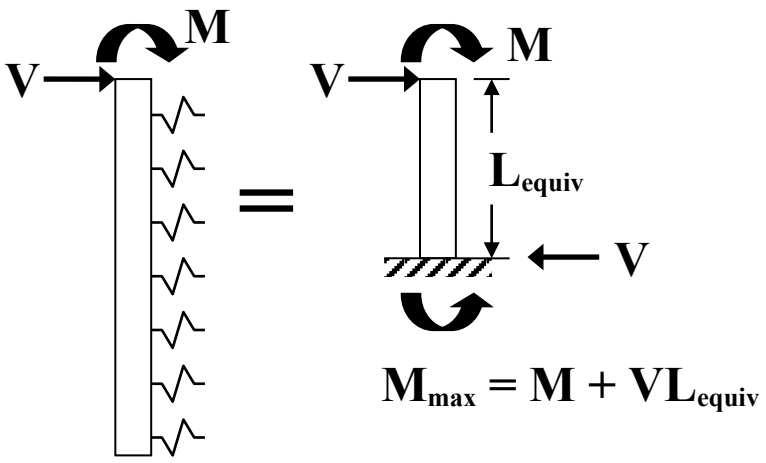


Figure 9-14 Equivalent Length Concept for Preliminary Design: (a) Nonlinear p-y Analysis; (b) Simplified, Point-of-Fixity Model

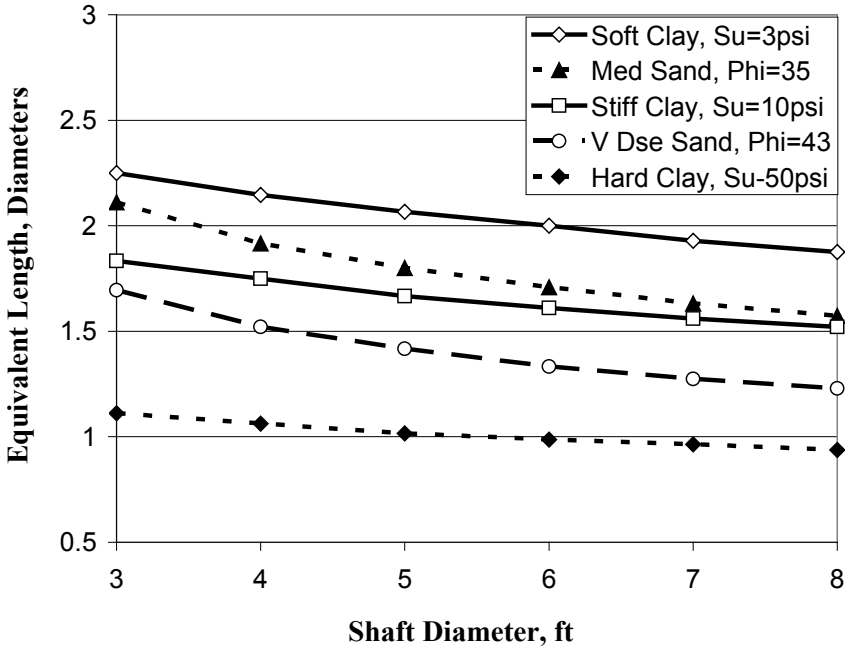


Figure 9-15 Equivalent "Fixity" Length of Shaft for Preliminary Design of Single Shaft Foundation (for estimating maximum bending moment only)

The values shown on Figure 9-15 have been computed for uniform deposits of clays ranging from soft to hard (undrained shear strengths of 3 to 50 psi) and for medium to very dense sands (φ ranging from 35° to 43°). This range of values should provide designers with a sense of the potential range in the magnitude of the maximum bending moments sufficient for planning or preliminary design purposes. Very hard materials such as rock would be expected to have an effective equivalent length for this purpose of less than one shaft diameter.

The equivalent drilled shaft lengths determined from Figure 9-15 will only provide an estimate of the maximum bending moment generated in the drilled shaft. This chart is for preliminary analysis of structural strength demand and should not be used to estimate drilled shaft displacement or rotation for serviceability conditions. An example of the use of this chart follows in Section 9.3.2.3.

9.3.2.2 Estimating Nominal Bending Moment Resistance

Structural design of drilled shafts is described in detail in Chapter 12 of this manual, and follows the provisions of AASHTO 5.6.4 for reinforced concrete columns. However, for selection of a trial shaft diameter in Block 10.2, it is helpful to make a preliminary estimate of nominal bending moment resistance.

For planning purposes, preliminary sizing of drilled shafts can be estimated using the following *approximate* values of nominal bending moment capacity:

1. For shafts with longitudinal reinforcing equal to approximately 1% of the gross cross sectional area,

$$M_n, \text{(k-ft)} \approx 27\ D^3 \qquad 9\text{-}1$$

2. For shafts with longitudinal reinforcing equal to approximately 1.5% of the gross cross sectional area,

$$M_n, \text{(k-ft)} \approx 40\ D^3 \qquad 9\text{-}2$$

Where:

D = Shaft diameter in ft.
M_n = The nominal bending moment resistance of a circular reinforced column with 3 inches cover and no axial compression load applied.

Since non-zero axial compression forces generally increase the nominal bending moment capacity for tension controlled members (strength conditions dominated by flexure rather than axial compression), the above estimates are likely to be conservative for most cases.

For structural design of the drilled shaft, the nominal strength in flexure (M_n) times a resistance factor (φ) must exceed the computed bending moments produced from the factored design loads, i.e. $M_{max} \leq \varphi M_n$. The resistance factor (φ) for columns under combined axial and bending (outlined in AASHTO 5.5.4.2) ranges from 0.75 to 0.9, with the value of 0.9 typically governing in those cases dominated by flexure. A complete discussion of drilled shaft design for structural strength is provided in Chapter 12.

9.3.2.3 Example of Preliminary Analysis for Lateral Loading

As an example of the use of the approach outlined above for preliminary selection of shaft diameter, consider a column foundation which is subject to a lateral shear force (factored load) at the ground line of 40 kips combined with an applied factored overturning moment of 800 kip-ft. The single drilled shaft will be embedded into a very stiff clay soil, with an undrained shear strength estimated to be 15 psi.

As a first estimate (Step 10.2), consider a shaft with a diameter of 4 ft. Figure 9-15 would suggest an equivalent "fixity" length of 1.5 to 1.7 diameters, or 6.0 to 6.8 ft, and thus an approximate maximum bending moment would be in the range of 800 k-ft + (6.8 ft)(40 k) = 1072 k-ft. A 4-ft diameter shaft with 1% reinforcing is expected to have a factored bending moment capacity of at least $(0.9)(27)(4)^3$ = 1555 k-ft, well in excess of the 1072 k-ft required (Step 10.4).

Using the 1072 k-ft as roughly equal to the required factored moment capacity, the minimum diameter can be estimated as $D = \{1072/[(0.9)(27)]\}^{1/3}$ = 3.53 ft. Therefore a shaft in the range of 3.5 ft to 4 ft diameter is likely to be sufficient and may be considered for complete design as described subsequently.

9.3.3 Computational Procedures and Design Methodology

To complete the design of a drilled shaft for lateral loading outlined in Blocks 10.3 through 10.5, the engineer must:

(a) compute geotechnical resistance to overturning of the shaft (Block 10.3),
(b) compute the maximum bending moment and shear force in the embedded drilled shaft (Block 10.4),
(c) compute the nominal moment capacity of the shaft (Block 10.4), and
(d) compute deflection and rotation at the head of the drilled shaft (Block 10.5).

The reaction provided by the soil or rock influences each of these steps in the process.

The recommended methodology for computing the response of drilled shafts to lateral and overturning forces is the "p-y method" which models the shaft as a nonlinear elastic beam and uses a series of nonlinear springs to model the soil resistance. This model has been found to capture the essential mechanisms of the problem, and has a history of successful use in both transportation and offshore foundation engineering. It requires the use of a computer, but available software is user-friendly and straightforward to apply, and there are several computer codes available for implementation of this model. A brief description of the p-y method is presented in the following section.

9.3.3.1 Brief Description of the p-y Method

The "p-y Method" represents a relatively sophisticated model which can effectively capture the nonlinear aspects of the problem, and is the recommended method for design of drilled shafts for lateral loading. This approach is readily implemented using one of several available computer software packages. An overview of the basic principles is provided below and described in more detail in GEC-9 (Parkes, et al., 2018).

The p-y method is a general method for analyzing laterally loaded piles and drilled shafts with combined axial and lateral loads, including distributed loads along the pile or shaft caused by flowing water or creeping soil, nonlinear bending characteristics, including cracked sections, layered soils and/or rock and nonlinear soil response.

The application of lateral load to a drilled shaft must result in some lateral deflection. This deflection causes a soil reaction that acts in a direction opposite to the deflection; i.e., the soil pushes back. The magnitude of the soil reaction along the length of the drilled shaft is a nonlinear function of the deflection, and the deflection is dependent on the soil reaction. The "p-y" method is so named because the soil

resistance is modeled as a nonlinear spring in which the force due to soil resistance, p, develops as a function of deflection, y, and the relations between the two are modeled as p-y curves.

A physical model for the laterally-loaded drilled shaft is shown in Figure 9-16. A drilled shaft is shown in the figure with loadings at the top. The soil has been replaced with a series of mechanisms that show the soil response in concept. At each depth, x, the soil reaction, p (resisting force per unit length along the drilled shaft), is a nonlinear function of lateral deflection, y, and is defined by a curve that reflects the shear strength of the soil, its Young's modulus, the position of the piezometric surface, the drilled shaft diameter, depth and whether the loading is static (monotonic) or cyclic.

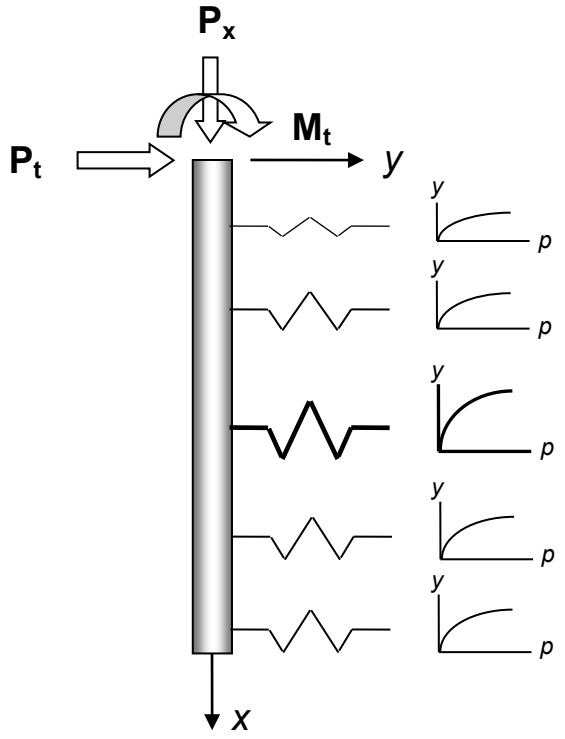


Figure 9-16 Model of a Drilled Shaft Foundation Under Lateral Loading Showing Concept of Soil Response Curves

The computational procedure is dependent on being able to represent the response of the soil by an appropriate family of p-y curves. Full-scale experiments and theory have been used and recommendations have been presented for obtaining p-y curves, both for static and for cyclic loading. Detailed descriptions of available p-y models are provided in FHWA-RD-85-106 (Reese, 1986). These models have been programmed as subroutines to computer programs, and the user merely needs to input the loadings, the section geometry of the drilled shaft and its stiffness, and the soil, steel and concrete properties. Other p-y methods can also be specified (e.g., Murchison and O'Neill, 1984, the API method for sands; Reese, 1997, an updated method for rock based on analysis of loading tests), and additional relations will likely be added as research and field experiments allow for their development. Site-specific p-y relationships as obtained from field loading tests can also be input by the user. Application of cyclic p-y criteria are important for situations in which the loading is cyclic, such as wave loading, wind loading and loading from seismic events, since the geomaterial will weaken compared to cases in which the loading is constant. Section 9.3.3.4 provides general design guidelines using currently available p-y models.

Section 9.3.3.5 provides guidelines on selection of the appropriate p-y model for a range of geotechnical conditions.

The computer programs provide an opportunity for quickly and easily investigating the influence of a large number of parameters. Some of these factors are the loading; the geometry, stiffness, and penetration of the drilled shaft; soil properties; and the interaction between the drilled shaft and the superstructure. In addition, if an unsupported portion of the drilled shaft extends above the groundline, buckling can be easily studied.

One of the most appropriate uses of the programs is to investigate the effects of drilled shaft penetration on performance. For a given system of loads, the penetration of the shaft can be varied and the lateral deflection of the head can be determined as a function of penetration. For "short" and "intermediate" length shafts, the lateral deflection will vary considerably with changes in penetration, but as the shaft becomes "long," further penetration will have essentially no effect on lateral displacement.

The presence of a tieback or strut can be simulated by means of a very stiff p-y curve, representing the axial stiffness of the support, at the location of the support. That curve is merely input along with the p-y curves for the soil. As briefly described in Section 9.3.6, p-y curves can also be offset by a specified displacement with respect to the drilled shaft, with the result that the soil resistance forces are mobilized against the drilled shaft in order to simulate an active pressure condition.

9.3.3.2 Simulation of Nonlinear Bending in Drilled Shafts

9.3.3.2.1 General

For design of drilled shafts under lateral loading, the engineer must recognize that the shaft is essentially a reinforced concrete beam-column and that its bending behavior cannot always be appropriately represented by a simple linear elastic beam, that is, by a single EI value. If the purpose of the analysis is to determine moments and shears within the shaft in order to design the reinforcing steel and to obtain the appropriate diameter, a linear analysis will almost always be sufficient. But, if the purpose of the analysis is to estimate deflections and rotations of the head of the shaft, nonlinear bending should be considered.

With regard to structural design of the shaft (covered in detail in Chapter 12), the amount and placement of the reinforcing steel must be sufficient to satisfy the moment demand for the structural limit state. Some bending moments are also caused by the unavoidable eccentricity of axial loads; however, such moments are dissipated within the top few diameters of the drilled shaft, even when the surface soils are relatively weak. When designing reinforcing cages, therefore, it is recommended that a method be used that will produce the moment and shear diagrams under the critical combination of factored loads, including axial loads applied at the bounds of the eccentricity permitted by the construction specifications. The reinforcing steel cage can then be designed rationally, including a reduction in the amount of longitudinal reinforcement with increasing depth of the drilled shaft. The p-y method is well-suited for this type of analysis.

9.3.3.2.2 Computation of Nonlinear Bending Stiffness.

Concepts: As the bending moment on any reinforced concrete section increases to the point at which it produces tensile stresses on one side of the shaft that exceed the tensile strength of the concrete, the section will crack, and a dramatic reduction in the EI of the section at that point will occur. Since the concentric axial component of load (if compressive) produces uniform compressive stresses across the section that superimpose upon the bending stresses, the moment at which cracking occurs is a function of

the magnitude of axial load on the drilled shaft. The assumption normally made is that cracks will be closely spaced along segments of the shaft in which the net tensile stress exceeds the tensile strength of the concrete.

Stress-Strain of Concrete and Steel: Nonlinear stress-strain curves are used for both steel and concrete. It is assumed that compressive collapse occurs in the concrete at the ultimate value of normal strain, ε_c, of approximately 0.003; for steel, the ultimate value of strain in both tension and compression is taken as 0.0020. The tensile strength of the concrete f_r, is taken as $7.5\sqrt{f_c'}$; here f_r, and f_c' (compressive strength) are both in units of lbs/sq.in. (psi), and the stress-strain behavior of the concrete in tension is assumed to be linear up to that stress.

Moment Curvature Relation: The derivation of the relation between bending moment, axial load and EI proceeds by assuming that plane sections in a beam-column remain plane after loading (Reese et al., 1998). Thus, when an axial load, P_x, and a bending moment, M, are applied, the neutral axis will be displaced from the center of gravity of a symmetrical section. The equilibrium equations for such a condition can be expressed as follows, where σ is a stress normal to the section.

$$b\int_{h_2}^{h_1} \sigma \, dy = P_x \qquad 9\text{-}3$$

$$b\int_{h_2}^{h_1} \sigma \, y \, dy = M \qquad 9\text{-}4$$

The terms used in the above equations are defined in Figure 9-17.

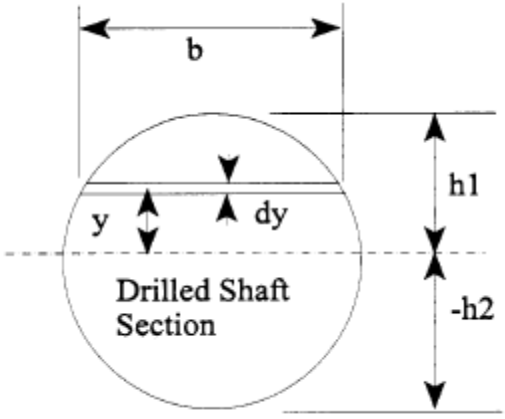


Figure 9-17 Definition of Terms in Equations 9-6 and 9-7

The numerical procedure for determining the relation between axial load, bending moment and EI of the section, considering the nonlinear stress-strain properties of the concrete and steel, is as follows for compressive axial loading and applied bending moment.

- A position of the neutral axis is estimated and a strain gradient, φ_ε, across the section about the neutral axis is selected. φ_ε is defined such that the product of φ_ε and distance from the neutral

axis gives the strain at a specific distance from the neutral axis. φ_ε has units of strain / length and is assumed to be constant, whether the section is in an elastic or an inelastic state. This defines the strain at every point in the section.

- Knowing the strain distribution across the section and the stress-strain relations for the steel and concrete, the distribution of stresses (σ) across the cross-section are computed numerically.
- The axial load acting upon the section is the integral of all of the compressive and tensile normal stresses acting on the section over the area of the section (Equation 9-3). If the value of computed axial load does not equal the applied axial load (P_x), the position of the neutral axis is moved and the computations are repeated. This process is continued until the computed value of P is equal to the applied value of P_x.
- The bending moment associated with this condition is then computed by summing moments from the normal stresses in the cross-section about a convenient point in the section (e.g., the centroidal axis or the neutral axis, Equation 9-4).
- The EI value for this particular stress state in the cross section, which is associated with particular values of axial load P_x and bending moment M, then remains to be determined.
- It can be shown from beam mechanics theory that $EI = M / \varphi_\varepsilon$, where φ_ε is the curvature (rate of change of slope) of the beam. Therefore, a unique relationship between P_x, M and EI is found for any particular section considering the number and placement of steel bars, the compressive strength of the concrete (and therefore its tensile strength) and the yield strength of the steel. The process is repeated for different values of φ_ε, so that a complete relationship between M and EI can be obtained for a given value of P_x. An example of such a relationship for two different values of P_x is given in Figure 9-18.

Figure 9-18 illustrates the effect of the axial load, P_x, on the relationship between M and EI. The presence of a compressive axial load stiffens the section by retarding the onset of cracking. In this example the EI at the nominal moment is also higher when the compressive axial load is applied.

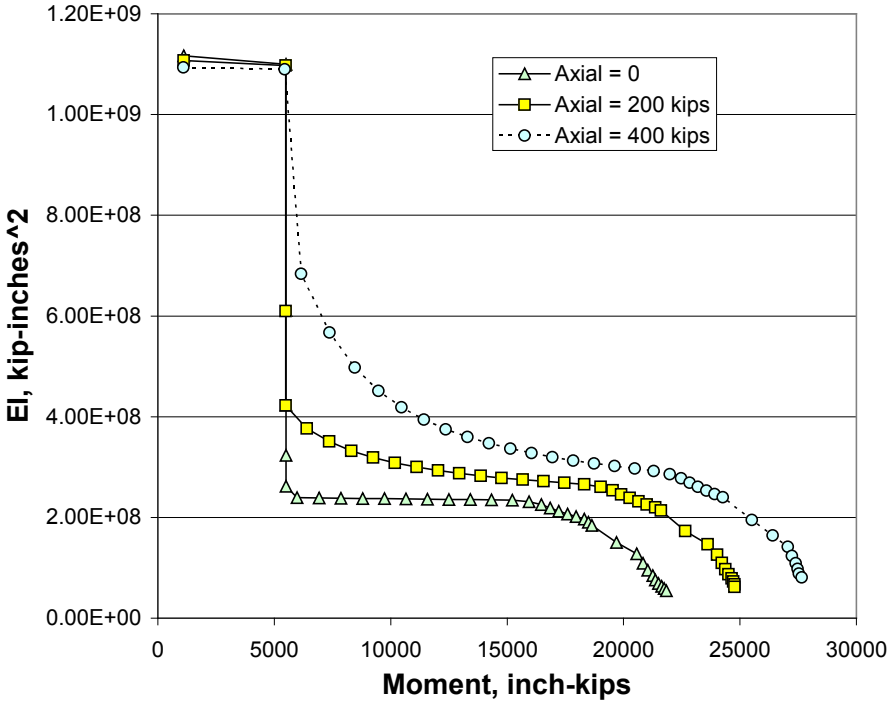


Figure 9-18 Variation of EI of a Drilled Shaft Cross Section with Bending Moment and Axial Load (4-ft diameter shaft with 12 #11 bars, 3 inches cover)

The stress-strain curves that are typically used in current software are shown in Figure 9-19 and Figure 9-20 for concrete and steel, respectively.

Referring to Figure 9-19:

$$f_c'' = 0.85 f_c' \tag{9-5}$$

$$E_c(initial\ slope\ of\ stress-strain\ curve) = 57{,}500\sqrt{f_c'} \tag{9-6}$$

$$f_c = f_c''[2(\varepsilon/\varepsilon_o) - (\varepsilon/\varepsilon_o)^2] \quad (\varepsilon < \varepsilon_o) \tag{9-7}$$

$$\varepsilon_o = 1.7 f_c' / E_c \tag{9-8}$$

$$f_r = 7.5\sqrt{f_c'} \tag{9-9}$$

In these equations, the units of E_c, f_c (axial stress), f'_c (28-day cylinder strength), and f_r (tensile strength) are in lbs/sq.in. (psi) and strain, ε, is dimensionless.

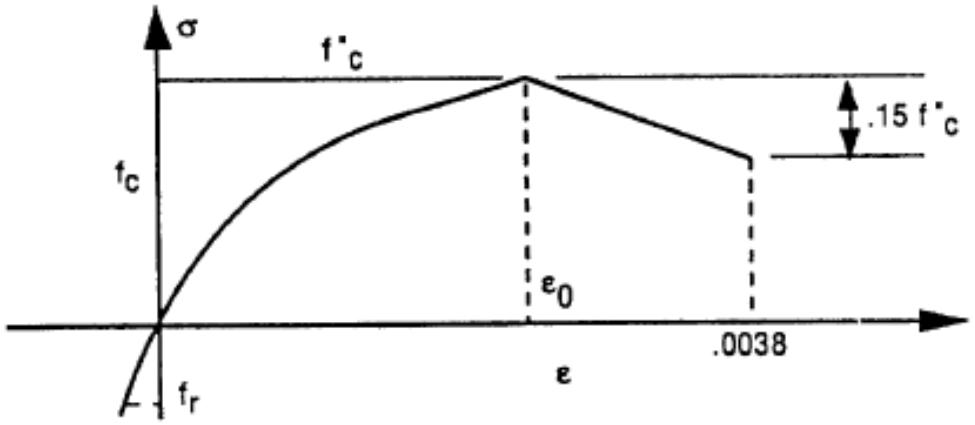


Figure 9-19 Stress-Strain Model for Concrete

In Figure 9-20:

$$\varepsilon_y = f_y / E \tag{9-10}$$

$$E = 29{,}000{,}000\ psi \tag{9-11}$$

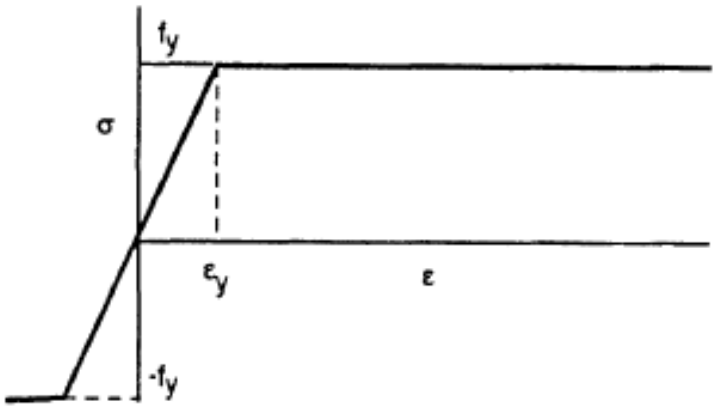


Figure 9-20 Stress-Strain Model for Steel Reinforcement

Software used to compute moments, shears and deformations in the drilled shaft using the p-y method contain subroutines that automatically perform these computations and adjust the EI value along the drilled shaft during the computation according to relationships similar to the ones shown in Figure 9-18. The user need only input the strength properties of the concrete and steel and the geometric properties of the cross section and longitudinal rebar. The deflected shape, shears and moments that are computed for the drilled shaft with a prescribed system of loads then reflect the effects of nonlinear bending, including cracking.

9.3.3.3 Design of an Individual Drilled Shaft for Lateral Loading

Using the principles outlined in the previous sections, the design of a drilled shaft for lateral loading is performed to satisfy the limit state conditions of stability (geotechnical limit state), flexural strength (structural limit state) and deformations (service limit state). Note that the design of the shaft diameter, depth of penetration, and structural reinforcing associated with each of these limit states must also consider axial loading and possibly other conditions as well.

9.3.3.3.1 Geotechnical Strength Limit State

The shaft must be of sufficient size and penetrate to sufficient depth to support the factored design loads for each possible load case without collapse. In computations of the geotechnical limit state, deflections are not a controlling consideration.

For analyses of geotechnical strength using the p-y method, the analysis of a trial shaft diameter and penetration depth is performed in the following manner:

1. The shaft is modeled as a simple linear elastic beam, with the elastic modulus equal to that of concrete (about 4,000,000 psi) and the moment of inertia equal to that of the uncracked circular cross section ($I = \pi D^4/64$). It is important that this simple model be used rather than the nonlinear stress-strain model for the purpose of geotechnical strength limit computations so that convergence of the analytical model is not limited by the flexural strength of the drilled shaft (which will be evaluated subsequently).
2. The soil is modeled using the best estimate of the governing soil conditions for the structure, as appropriate for each load case. Where a number of separate foundation units are designed using

a similar model, the governing soil conditions used in the model should reflect the least favorable conditions at any individual foundation location within the zone for which the model is used. Separate models should be developed where significant variations in ground conditions or loadings are anticipated (Block 10.1 of the design process flow chart in Figure 9-13). Local scour is considered, as required, for a specific load case.

3. The load at the shaft head is applied in various multiples up to and exceeding the factored design load to compute deflections and thus perform a type of "pushover analysis" (Block 10.3). An unstable condition will have the result that the computer software is unable to converge to a solution, or else the solution converges to an extremely large head deflection.
4. Although deflection is not the controlling consideration for stability, the computed deflection must be a reasonable value (10% of the shaft diameter) at and slightly larger than the factored design loads to ensure that the shaft is of sufficient size and depth that geotechnical strength requirements are satisfied.
5. Strength at loads higher than the factored design load is necessary to ensure that a ductile lateral load response exists and that there is adequate reserve strength to accommodate site variability and other contingencies. This reserve strength is achieved by including a resistance factor less than one on the geotechnical strength for lateral resistance. AASHTO Table 10.5.5.2.4-1 currently (2017) provides for a resistance factor of 1.0 for nominal horizontal geotechnical resistance of a single shaft or shaft group and for extreme limit states. However, the values in Table 9-1 are recommended for design for geotechnical strength considerations.

TABLE 9-1 RECOMMENDED RESISTANCE FACTORS FOR GEOTECHNICAL STRENGTH LIMIT STATE FOR LATERAL LOADING OF DRILLED SHAFTS (COMPUTED USING P-Y METHOD)

Condition	Resistance Factor, φ
Overturning resistance of an individual or single row of elastic shafts, free to rotate at the head	0.67
Pushover of an elastic shaft within a multiple row group, with moment connection to the cap	0.8
Extreme event strength cases	0.8

The values recommended in Table 9-1 should be considered suggested values based on the authors' judgment, in recognition that this recommended approach provides a check for ductility and geotechnical stability that exceeds the level of reliability provided by the current AASHTO (2017a) code provisions.

For this evaluation of geotechnical strength, it is recommended that the pushover analysis model of the shaft should converge to a solution at loads that are at least 1/φ times the factored design load for each strength limit state load case. Satisfactory convergence would be indicated by computed deflections that are no larger than 10% of the drilled shaft diameter. In many cases, the computed flexural strength of the column or connection of the shaft to the structure will be insufficient to transfer 1/φ times the factored design load. Likewise, the computed flexural strength of the shaft may be insufficient at loads which are

1/φ times the factored design load. However, the geotechnical strength analyses described above are performed with the shaft modeled as a linearly elastic beam only to ensure that adequate embedment is achieved to provide strength and ductility from the lateral soil resistance acting on the shaft.

After the basic model parameters (soil profile, loads, etc.) are input, repeated trial analyses of various shaft diameters and embedded lengths can be performed quickly and easily using available software to determine a minimum shaft diameter and embedded length. If the shaft diameter is controlled by other factors (e.g., column size or anchor bolt pattern), the minimum embedded length for lateral loading can be determined for the defined shaft diameter. Note that design for axial or other considerations may ultimately dictate greater shaft diameter or embedded length.

9.3.3.3.2 Structural Strength Limit State

The drilled shaft must be of sufficient size and adequately reinforced with both longitudinal and transverse reinforcement to resist the combination of bending, shear, and axial stresses which are imposed by the design loads. Therefore, the nominal axial, shear, and flexural resistance of the shaft cross section must exceed the factored axial, shear, and bending moments. At this point in the design process (Block 10.4 in Figure 9-13) where the minimum diameter and depth for lateral loads is established, only a preliminary check of the structural strength limit state is performed. The structural design of the shaft is finalized in Block 12 of Figure 8-1 along with the structural design of the drilled shaft connection to the structure. The forces computed as a result of analyses for lateral loading are likely to control the structural design of the shaft.

The analyses performed for geotechnical strength using a simple linear elastic shaft will provide computed values of bending moment in the shaft at the factored design loads. The maximum bending moment in the shaft is a function of the applied loads and the soil resistance distribution along the length of the shaft; the nonlinear bending stiffness (EI) described in Section 9.3.3.2.2 has relatively little effect on the maximum computed bending moments. A quick check of potential bending moment resistance for a given shaft diameter can be made using the preliminary estimates of bending moment resistance outlined in Section 9.3.2.2. If the computed maximum bending moments greatly exceed the preliminary estimate of resistance for a shaft with reinforcing equal to approximately 1.5% of the cross sectional area, the designer should consider a larger shaft diameter.

Although the complete structural design must consider combined axial, shear, and bending, the first check of required longitudinal reinforcement for flexure can be determined at this point using the concepts described in Section 9.3.3.2.2. The complete structural design procedure includes the following general considerations:

- The axial, shear, and bending moments are determined using factored loads.
- The longitudinal reinforcing is determined which provides a nominal structural strength exceeding the combined factored axial forces and bending moments in the shaft for each load combination.
- The nominal shear resistance of the concrete section is compared with the factored shear forces, and additional transverse reinforcement (above the code-specified minimum) is determined, if necessary.

The complete structural design of the drilled shaft is described in Chapter 12.

9.3.3.3.3 Serviceability Limit State (Lateral Deformations)

Deformation limits should be chosen based upon actual serviceability requirements for the structure rather than "rule of thumb" criteria. Deformation and rotation computed at the top of shaft for a single column, single shaft foundation can be amplified at the pier cap for a tower bent. Structures such as overhead signs or sound walls are not particularly sensitive to deformations, but some serviceability limits may be established for aesthetics, functionality, or other considerations. Some extreme event load cases (for example, the check flood for scour or some seismic loadings) may not include serviceability requirements.

In many cases, a significant component of the deformation can be related to flexural stiffness of the drilled shaft and not the ground response. The nonlinear model of the reinforced concrete shaft in flexure described in Section 9.3.3.2.2 should be used to model this condition. Service loads are used in the analysis. With the computer model that has been established for the analysis of strength, this step generally only requires the adjustment of the load for the service limit state conditions and the additional input to specify the nonlinear structural material properties of the shaft (longitudinal reinforcement and concrete strength and modulus).

9.3.3.3.4 Example

Consider the example used in the preliminary analysis, which was a drilled shaft with a factored shear force at the ground line of 40 kips and a factored overturning moment of 800 kip-ft. Borings have been made which suggest that the soils consist of a very stiff clay soil having an undrained shear strength of 15 psi. Our preliminary analyses suggested that a 4-ft diameter shaft would likely be suitable. Maximum service loads have been determined to be 25 kips shear plus 500 kip-ft applied moment, and the maximum lateral deformation at the top of shaft is not to exceed 1/2 inch.

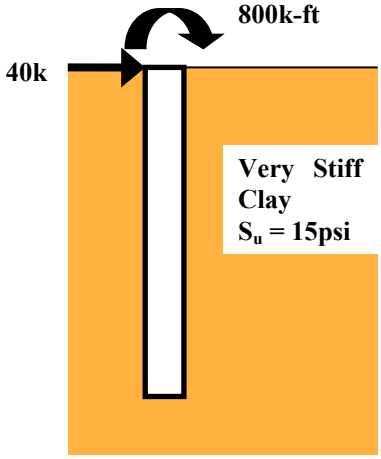


Figure 9-21 Simple Lateral Load Example, Factored Loads Shown

Geotechnical Strength Limit State:

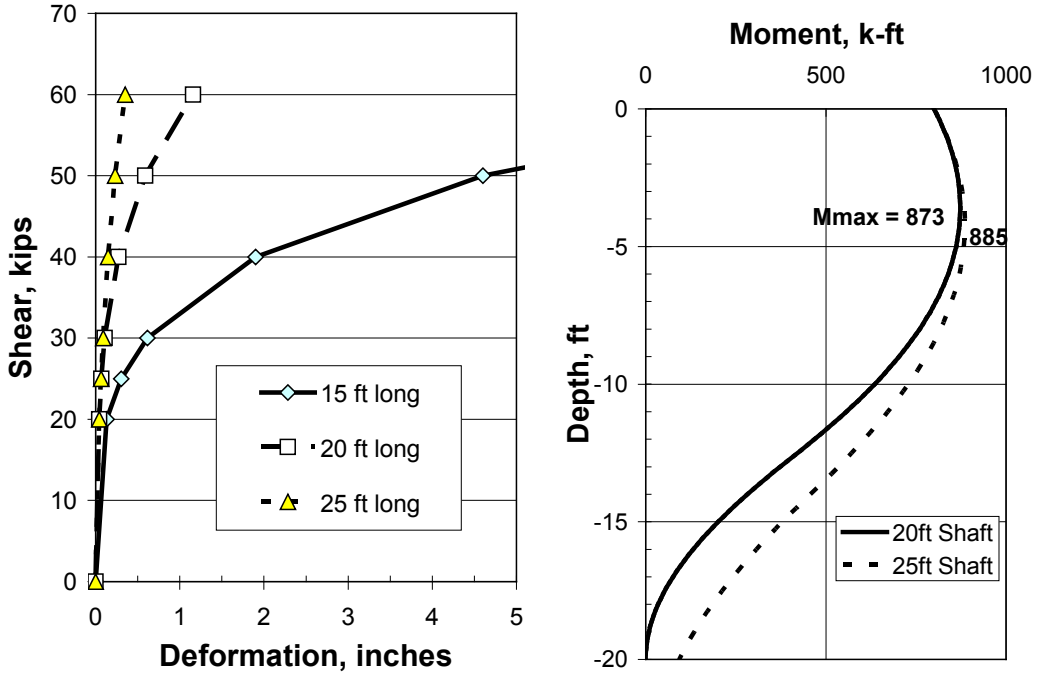
For this simple problem, the computer software LPILE® is used to perform analyses with the p-y method on a 4-ft diameter shaft with a range of loads (all with applied moment and shear in proportions similar to the factored design loads of 40 kips shear + 800 kip-ft moment) and for several different embedded

lengths of shaft. For analysis of this problem using the computer software, the important input parameters are:

- Boundary conditions at top of shaft (applied shear and moment; axial force = 0)
- Trial shaft diameter (48 inches) and bending stiffness, EI (E = 4,000,000psi, I = $\pi d^4/64$ = 260,576 in^4)
- Trial shaft length, divided into 100 nodes for analysis
- Soil strength and stiffness (described in Section 9.3.3.4.1, for this simple one-layer problem: unit weight = 0.07 pci, cohesion = 15 psi, ε_{50},= default value of 0.005 for this soil)

The description of the problem provided above accomplishes Blocks 10.1 and 10.2 of the flow chart in Figure 9-13. For the analysis of geotechnical strength (Block 10.3), the guidelines provided previously in Table 9-1 suggest the use of a resistance factor, φ, of 0.67. To mobilize a resistance that, when multiplied by 0.67, meets or exceeds the factored load combination, it is necessary to perform the analysis using 1/φ times the factored loads. Therefore, the computer analysis of the trial shaft with linear elastic properties in flexure should converge to a solution with a deflection not exceeding 10% of the shaft diameter (= 4.8 inches) at applied lateral force and overturning moment which are 1.5 times (1/0.67) the factored loads, or 60 kips shear + 1200 kip-ft overturning moment.

The results of these analyses in terms of top of shaft deformation versus shear (and applied moment) are illustrated on Figure 9-22a for a range of shear + overturning at the top of shaft of up to 60 kips + 1200 kip-ft applied overturning moment. The graph is presented with shear on the vertical axis, but each data point is generated using applied overturning moment in proportion to the shear.



(a) (b)

Figure 9-22 Results of Analyses of Geotechnical Strength for Simple Example: (a) Shear Load versus Deformation; (b) Bending Moment versus Depth

The analyses indicate that marginal stability is achieved with a 15 ft deep shaft as evidenced by the large deformation at a load which is 1.5 times (1/φ) the factored design load. Although a convergent solution was obtained, the computed deformation of this shaft at this load exceeds the general guideline recommendation for the geotechnical strength limit condition of 10% of the shaft diameter (4.8 inches). By contrast, an additional 5 ft of embedment provides a much more robust design with stability at loads in excess of the factored design load.

The maximum bending moment is not much affected by embedded length as is evident from the results of the aforementioned analysis for the 20 and 25-ft long shafts plotted on Figure 9-22b. The maximum computed bending moment for the factored load case (40 kips shear + 800 kip-ft moment) is about the same for each, around 880 kip-ft. Note that this maximum moment is somewhat less than the 1072 k-ft computed using the preliminary approximate solution.

For the next step in the design process, the 20-ft long by 4-ft diameter shaft is considered for further analyses.

Structural Strength Limit State and Serviceability Limit State:

Evaluation of structural limit state (Block 10.4) and serviceability limit state (Block 10.5) can be performed by revising the analysis to include nonlinear flexural stiffness. Analysis is performed by specifying the shaft to be a nonlinear reinforced concrete circular member, 48 inches diameter, and reinforced with 12 #11 bars (equal to approximately 1% of the cross sectional area of the concrete) equally spaced in a circular arrangement with 3 inches cover. Concrete compressive strength of 4,500 psi and reinforcement yield strength of 60,000 psi is specified.

Results of analyses for structural and serviceability limit states are illustrated on Figure 9-23.

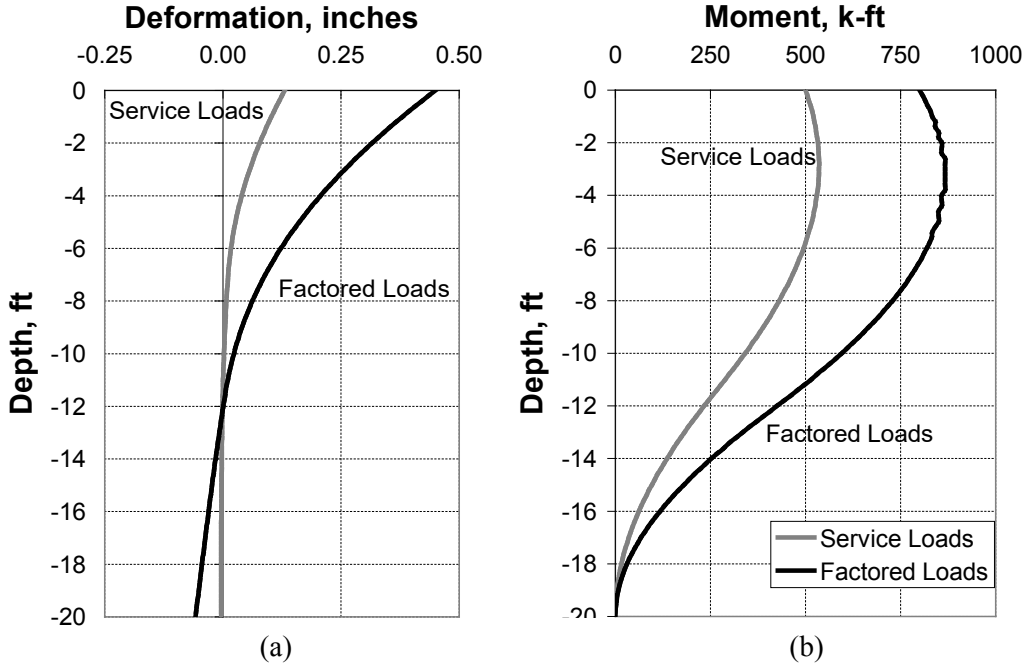


Figure 9-23 Results of Analyses of Structural Strength and Serviceability for Simple Example: (a) Deformation versus Depth; (b) Bending Moment versus Depth

The computed maximum bending moment for the analysis of structural strength using factored loads and nonlinear EI for the shaft is 865 k-ft (Figure 9-23b), compared to 873 k-ft for the linear elastic model at the same magnitude of load (shown on Figure 9-22b). As noted previously, the computed maximum bending moment is not very sensitive to the flexural stiffness of the shaft.

The maximum computed deflection was 0.13 inches at the top of shaft for the service load condition of 25 kips shear, 500 k-ft applied moment (Figure 9-23a). This value is well within the stated requirement for this example of 0.5 inches deflection at service loads. Note that the computed deflection at this magnitude of lateral and overturning force for the earlier analysis with constant (linear elastic) EI was 0.07 inches. The larger deflection for the nonlinear case is due to reduced flexural stiffness in the shaft caused primarily by cracking of the drilled shaft concrete.

The computed relationships of bending moment vs. curvature and bending moment vs. flexural stiffness (EI) for this tentative design is computed using the procedures outlined in Section 9.3.3.2.2 and the results of these analyses are illustrated on Figure 9-24 and Figure 9-25, respectively. The computed nominal moment resistance (labeled "M_{ult}" on Figure 9-24 and Figure 9-25) is significantly larger than the computed moment from the factored design load.

Summary:

In summary, this simple example illustrates the process of design to include each of the three important limit state conditions: 1) geotechnical strength limit state (diameter and embedded length), 2) structural strength limit state (shaft diameter and longitudinal reinforcing), and 3) deflections for serviceability limit state. The complete design will require consideration of all other loading conditions (particularly axial loads). In some cases, groups of drilled shafts may be considered, as described in Chapter 11.

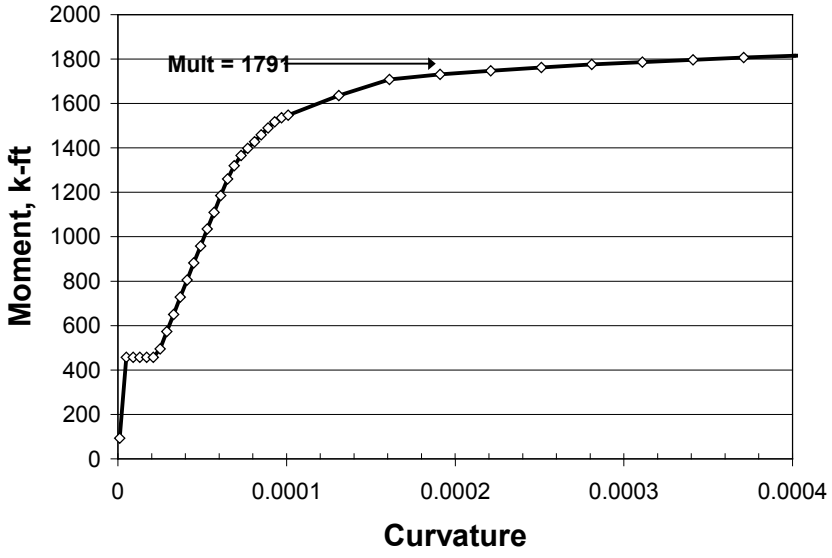


Figure 9-24 Bending Moment versus Curvature for Simple Example

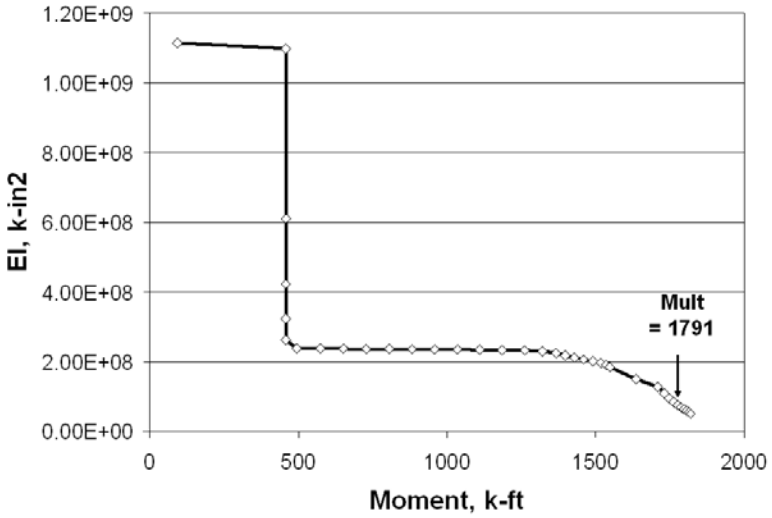


Figure 9-25 EI versus Bending Moment for Simple Example

9.3.3.4 Guidelines for Selection of Appropriate p-y Criteria for Design

This section provides an overview of available p-y criteria used in design of drilled shafts for lateral loads. The purpose of this discussion is to provide some guidance in the appropriate selection and use of these criteria for typical problems in design of transportation structures. A more complete description of the experimental basis and applicability for each criterion is available through the provided references in each section and through technical manuals associated with specific software codes.

The p-y criteria are simply a means of associating the soil resistance mobilized as a nonlinear function of displacement at various points along a drilled shaft. Although there may exist a theoretical basis in many cases, the criteria used in design are empirical in that the final form of the models used are derived from experiments (instrumented load tests). Therefore, it is necessary that the user understand the experimental basis for a p-y criterion that is being used so that limitations are understood and for the models to be used for appropriate conditions.

9.3.3.4.1 Cohesive Soils

Several criteria are available for modeling cohesive soils, and the most commonly used include those for soft clays (Matlock, 1970), stiff clays (Welch and Reese, 1972) and stiff clays in the presence of free water (Reese et al., 1975). Each of these are characterized by the use of a polynomial to model the nonlinear relationship of soil resistance versus displacement followed by an upper bound, as shown in Figure 9-26.

The input soil parameter that most significantly affects the response of a drilled shaft in cohesive soils using p-y curves for these criteria is the soil cohesion (undrained shear strength, s_u), which directly affects the ultimate soil resistance, P_u. The undrained shear strength used to develop the p-y criteria is typically measured using unconsolidated, undrained (UU) triaxial compression tests with confining pressures at or near the total overburden pressure.

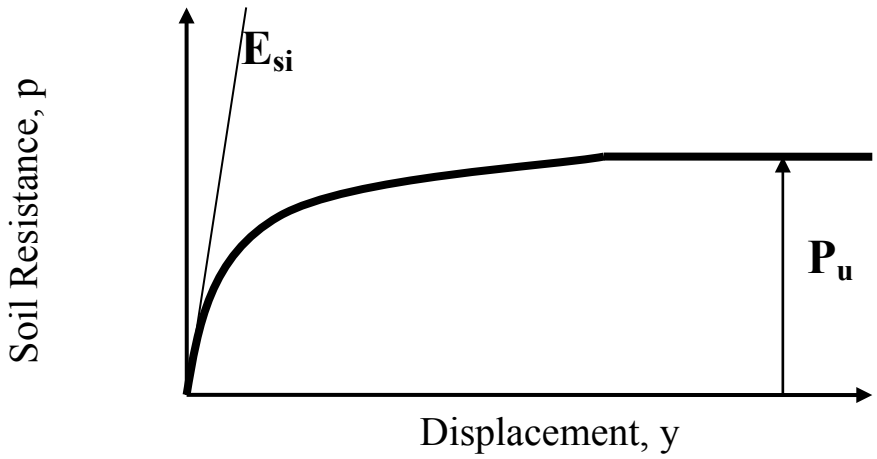


Figure 9-26 Conceptual p-y Curve for Cohesive Soil

A parameter which has a somewhat less significant effect in cohesive soils is the initial stiffness, E_{si}. E_{si} is most commonly related to a stiffness parameter, ε_{50}, which is intended to represent the strain at an axial compressive stress equal to 50% of the yield stress in the UU triaxial test. Typical values of ε_{50} are often simply associated with a given range of s_u.

The use of undrained shear strength, s_u, has proven to provide a reliable correlation with load test results of short duration. The instrumented field loading tests performed to develop these criteria have typically been performed within a period of a few hours, so this model of soil resistance is appropriate for short duration loadings typical of live loads on highway structures. Long duration sustained loads may actually mobilize a softer response due to creep, and extremely short duration transient loads may actually mobilize a stiffer response due to rate of loading effects.

Engineers using these p-y criteria to design a drilled shaft for lateral loading should perform analyses using a range of values of s_u and ε_{50} to evaluate the sensitivity of the analysis to these parameters. There always exists uncertainty in the evaluation of in-situ soil properties as well as in the relationship of these properties to the ultimate performance of the foundation. Sensitivity studies can provide the information needed to develop judgment regarding the reliability of the design and the relative importance of various input parameters.

With respect to stiff clays, engineers are sometimes faced with the decision regarding the applicability of the criterion from Reese et al. (1975) for stiff clay in the presence of free water. The tests performed to develop this criterion were performed using cyclic loading at a site of stiff fissured clay ($s_u \approx 2$ ksf) in a submerged condition. The soil response was observed to rapidly degrade with multiple cycles of load due to localized scour adjacent to the pile, and the criterion developed for static loading also exhibits significant strain-softening resulting in a characteristic response as indicated in Figure 9-27.

The criterion shown in Figure 9-27 will result in a substantial reduction in mobilized soil resistance compared to that of Welch and Reese (1972), which does not include such strain softening. This reduction is only appropriate for situations where stiff clay is exposed to free water at or near the ground surface (such as drilled shafts installed at a water crossing), where degradation similar to that observed in the load test experiment can occur. In conditions where the groundwater surface is at depth and free water is not present at or near the ground surface, the Welch and Reese criterion is more appropriate, even

below groundwater. Similarly, stiff clay strata at depth below a sand stratum would normally not be subject to degradation due to free water (unless scour removed the overlying sand).

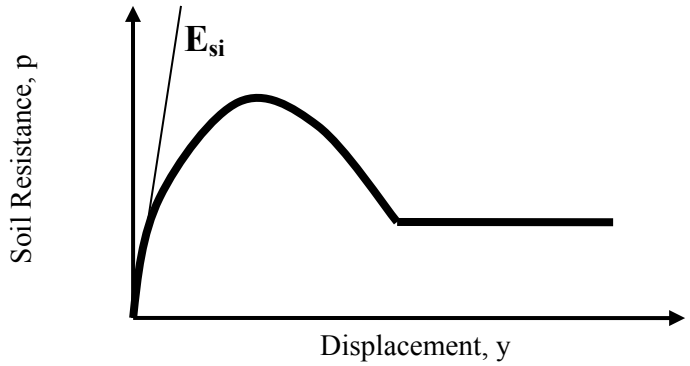


Figure 9-27 Conceptual p-y Curve for Stiff Clay in the Presence of Free Water

9.3.3.4.2 Cohesionless Soils (Sands)

Several criteria are available for modeling cohesionless soils, and the most commonly used include Cox, et al. (1974) and the very similar criterion of Murchison and O'Neill (1984) that has been adopted as a standard by the American Petroleum Institute. Each of these are characterized by an initial linear stiffness followed by a polynomial to model the transition to an upper bound, as shown in Figure 9-28.

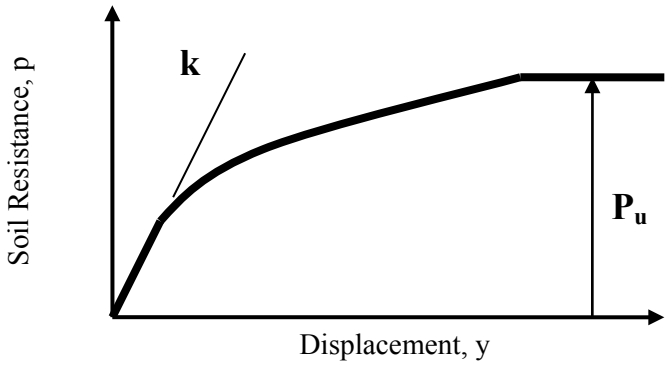


Figure 9-28 Conceptual p-y Curve for Cohesionless Soil

The input soil parameter that most significantly affects the response of a drilled shaft in sand using p-y curves for these criteria is the modulus value, k, which directly affects the initial straight line portion of the curve. The values used for this parameter k are estimated within the range of 20 to 225 lb/in.^{3} based on an assessment of the relative density of the sand and the effect of a submerged or dry condition. The ultimate resistance, P_u, is related to the angle of internal friction (φ) and the confining pressure, but the lateral response of drilled shafts in sand is less sensitive to φ than to k.

9.3.3.4.3 Cohesive Granular (c-φ) Soils

The c-φ criterion currently available in some commercial software is based on the superposition of p-y criterion for cohesive soils and cohesionless soils; this criterion is not supported by experimental data and

not recommended for foundation design. In the absence of site-specific load test data with which to calibrate an alternative p-y criterion, the designer should use an approach based on either a cohesionless soil or a cohesive soil with a best estimate of undrained shear strength as a function of depth.

Some recent research, including full scale load tests in cemented granular soil (loess), has been conducted in Kansas, and used to develop a p-y curve criterion for loess as reported by Johnson, et al. (2007).

9.3.3.4.4 Rock

The methods currently available for predicting the p-y response of rock are based on a limited number of experiments and on correlations that have been presented in the technical literature (Reese, 1997). In general, the recommendations provided for weak rock are recommended as conservative estimates for most design purposes.

The p-y response is represented by a curve (shown in Figure 9-29) similar to that used for cohesive soils described previously, except that the ultimate resistance is correlated to compressive strength, q_u, rather than undrained shear strength. In lieu of the stiffness parameter, ε_{50} used for cohesive soils, a similar parameter, k_{rm}, is used for weak rock. The initial stiffness, K_{ir}, of the curve is proportional to the compressive strength of the rock, the initial modulus of the rock mass, E_{ir}, and the value of k_{rm}, with suggested values of k_{rm} in the range of 0.00005 to 0.0005. The lower value provides a somewhat stiffer initial response.

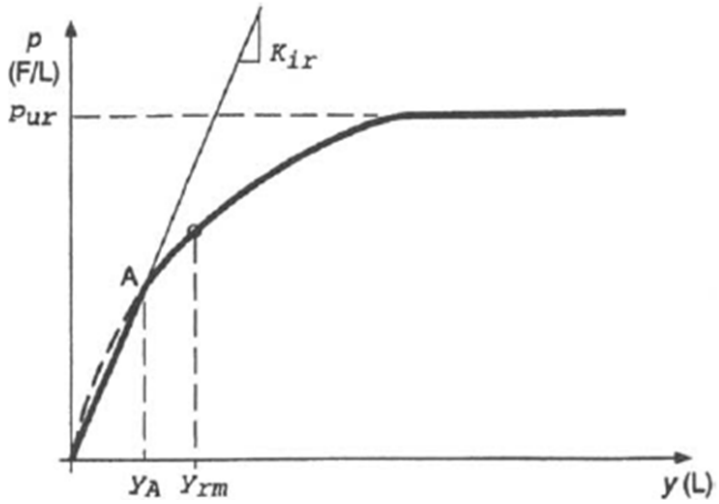


Figure 9-29 Conceptual p-y Criterion for Weak Rock (Reese, 1997)

In many cases, weak or decomposed rock is difficult to characterize in terms of strength and stiffness. A recent series of tests were performed of drilled shafts of various sizes and lengths socketed into a severely weathered and decomposed quartzite (Brown and Kahle, 2002). The decomposed rock had SPT N-values that were at 50 blows with less than 2 inches penetration, coring recovery values of less than 20%, and RQD values of 0. The test shafts, 3 to 5-ft diameter and embedded 1 to 2 diameters in rock, were loaded to geotechnical failure in overturning. The results of the load tests were modeled very effectively using the weak rock criterion with values of q_u = 180 psi, E_{ir} = 27,000 psi (150 q_u), and k_{rm} = 0.0004. These results might represent a reasonable lower bound estimate for cases in which hard but decomposed rock is known to be present when the geotechnical investigation is unable to provide much meaningful strength information about the decomposed rock formation.

Strong rock has been characterized using a model developed from tests in vuggy limestone of southern Florida, and the p-y response is modeled using a bilinear model as shown on Figure 9-30 (Ensoft, Inc., 2004). On that figure s_u is defined as 1/2 the unconfined compressive strength, q_u, and b is the diameter of the drilled shaft.

One distinctive characteristic of this p-y formulation is the fact that the field load tests upon which the criterion was based were carried to only limited displacement. The authors therefore thought it prudent to assume that a brittle fracture might occur at higher displacements, since the vuggy limestone is known to be quite brittle in shear. This upper limit at which brittle failure occurs works out to a shaft displacement at the location of the rock p-y curve equal to 0.24% of the shaft diameter; e.g. about 0.1 inches for a 3.5-ft diameter shaft and 0.2 inches for a 7-ft diameter shaft. At displacements larger than this upper bound value, the resistance of the rock is assumed to drop to zero. As a result of this assumed brittle behavior, an engineer using this criterion can find the curious result in which the computed lateral response of a drilled shaft is weaker using the "strong rock" criterion than would be the case using the "weak rock" criterion with similar strength values.

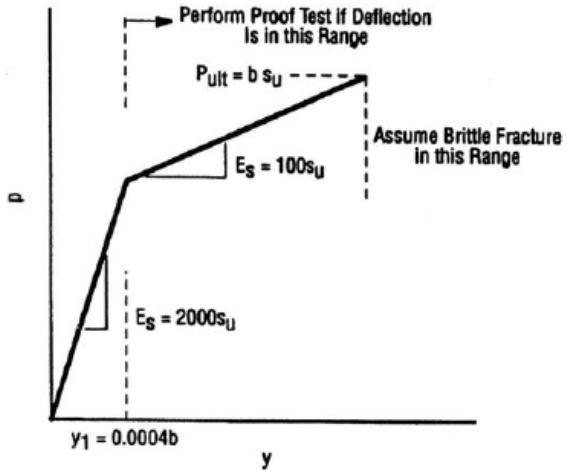


Figure 9-30 Proposed p-y Criterion for Strong Rock (Ensoft, Inc., 2004)

While there may be some extremely brittle rock formations in which strength and lateral resistance dramatically drops to near zero at a displacement in the 0.1 to 0.2 inch range, it is unlikely that this type of response is representative of most rock formations encountered in practice. It is recommended that the weak rock criterion be used for cases where shear failure of the rock mass is considered possible.

9.3.3.4.5 Correlations with *In-Situ* Tests

There exist a number of published correlations for computing p-y curves based upon the results of *in-situ* tests such as the pressuremeter (Briaud, 1992) or dilatometer (Robertson et al., 1985). In general, these test data rely on empirical correlations with test measurements to develop p-y curves for design, with consideration of soil type, depth, and shaft diameter. Comparative evaluations suggest that these tools can provide useful correlations (Anderson et al., 2003); as with any empirical relationship, calibration to field load tests in geologic conditions which are representative of the local area provide the most reliable application of these correlations for design.

9.3.3.4.6 Variations in Stratigraphy

In many cases, variations in stratigraphy across a site represent a more significant variable than the strength characteristics of any one particular stratum. This is particularly true when there exists a relatively soft soil layer overlying a strong layer such as rock.

As an example, consider the case shown in Figure 9-31 of a 6-ft diameter by 40-ft long drilled shaft installed through a 20-ft thick layer of soft clay (s_u = 2 psi) into a weak rock (q_u = 180 psi, E_{rm} = 27,000 psi) and loaded with a shear force of 75 kips at the ground surface. For this simple example, the effects of variations in soil properties are as follows:

Case	Deflection (in)	Maximum Moment (in.-kip)
Base Case (described above)	0.116	15,722
Soft clay thickness increased 10%	0.142, incr 22%	16,590, increase 5.5%
Soft clay strength reduced 10%	0.118, incr 2%	15,880, increase 1%
Rock q_u and E_{rm} reduced 10%	0.118, incr 2%	15,720, no change 0%
Strong Rock bearing, q_u = 750 psi	0.125, incr 8%	15,410, decrease 2%

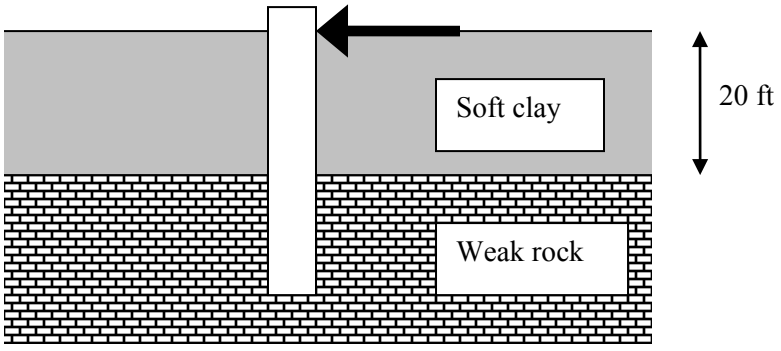


Figure 9-31 Example for Sensitivity Analysis of Stratigraphy

The results of this hypothetical problem indicate the relative importance of defining the stratigraphy and the possible range of variations with respect to lateral load response. It also demonstrates the value of performing a sensitivity analysis to determine the most important parameters. Note also that the use of the strong rock (vuggy limestone) criterion resulted in a larger computed displacement even with a q_u of 750 psi instead of the 180 psi used for the weak rock criterion. This counterintuitive result suggests that the use of the strong rock criterion might have limitations for use in rock formations beyond the vuggy limestone from which it was derived.

9.3.3.4.7 Effects of Cyclic Loading

Repeated cyclic lateral loading can affect the lateral soil resistance (most significant in cohesive soils), and a model of the degradation of soil resistance is included in the p-y criteria described previously. The most significant degradation effect occurs in submerged conditions when large lateral displacements at the soil surface result in gapping around the shaft. With repeated cycles of loading, water is alternatively sucked into the gap around the shaft and then expelled as the gap closes, eroding the soil and enlarging the gap. This effect can be a significant consideration for coastal structures subject to repeated wave loading.

9.3.3.4.8 Effects of Rate of Loading

Transient loads of short duration such as seismic, wind, and vessel impact loadings apply lateral loads to the soil at a higher rate of loading than is commonly measured during a static load test which may be conducted over a period of hours. The p-y criteria described previously have been developed based on static load tests and thus do not directly address the effect of a high rate of loading associated with transient load events. The higher strain rate associated with rapid lateral loading in soil would generally be expected to produce a somewhat stiffer response and greater strength. The use of static analyses may be somewhat conservative for such conditions, but the direct consideration of rate of load effects is generally not justified for routine foundation design.

Long term sustained lateral loads, such as from lateral earth pressure loads, may have the opposite effect. Particularly with respect to clay soils, long term creep can reduce the soil stiffness relative to a short term static p-y curve based on undrained shear strength.

9.3.3.5 Considerations for Scour and Extreme Event Loading

Extreme event load combinations may include forces from earthquakes, ice, vessel impact, and increased scour effects from the check flood on structure stability. Scour for the 100 year flood event is not considered an extreme event, but requires some special considerations. In addition, some extreme events are combined with different levels of scour (for example, vessel impact loads). In general, the recurrence interval of extreme events is thought to exceed the design life of the structure.

Ice and vessel impact forces are included in design using computational procedures described previously for static loading. In general, there are no special considerations for these load cases with respect to the computation of lateral load response other than the consideration of various combinations with different scour levels.

Scour is an important consideration for bridges over waterways, and the effects will lower the surface elevation of the subsurface profiles defined in Block 10.1 for design. Scour includes the general scour (scour that may occur without the presence of the structure) and channel contraction scour (due to the presence of the structure in the waterway) plus local scour immediately around the bridge piers. A further discussion of scour associated with the design flood event and the effects on the overall design of drilled shaft foundations is provided in Chapter 10.

The consequences of changes in foundation conditions from the design flood event are included in design for lateral loads at strength and service limit states. The scour associated with the check flood is considered an extreme condition which may lower the surface elevation more significantly for strength load combinations. AASHTO Section 10.5.5.3.2 also requires that the nominal resistance remaining after the check flood condition must be adequate to support the unfactored strength limit state loads with a resistance factor of 1.00.

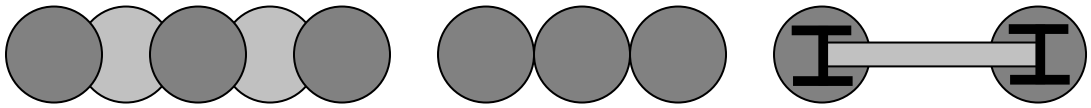
Earthquake-induced lateral loads imposed from the structure onto the foundation are typically analyzed using the computational procedures described previously for static loading, but may include some additional considerations. A discussion of the overall design of foundations for earthquakes is provided in Section 9.4.

9.3.4 Alternative Models for Computation of Shaft Response

Although the p-y method is recommended for design, available alternative models can be employed in some circumstances. The Broms Method (Broms, 1964a, 1964b, and 1965), offers a simple computation method that may be useful for sign or sound wall foundations constructed using short drilled shafts. In addition, the Broms Method provides a rational limit equilibrium solution which is easy to understand in terms of the basic principles of computing the strength limits of a simple problem. Several other alternative models include those based on elastic continuum, boundary element, and finite element models. An overview of these models is included in GEC-9 (Parkes, et al., 2018).

9.3.5 Design of Drilled Shaft Walls

Drilled shaft walls may be constructed as overlapping secant shafts, as tangent shafts, or as a soldier pile (shaft) wall (Figure 9-32). A soldier pile wall has space between the shafts that may or may not contain lagging or panels; if the shafts are spaced relatively closely together, the soil may arch between shafts and may be covered with a surface panel or shotcrete to provide a surface finish and protection from erosion. Secant shafts include combinations of primary/secondary shafts, in which the primary shafts are constructed of unreinforced concrete, followed soon after by the secondary (reinforced) shafts. In some cases, the primary shafts may be constructed using lower strength concrete. Tangent shafts are normally used in situations where groundwater is less of a concern, since the space between shafts is not sealed and seepage through the joint may occur. All types of drilled shaft walls can be covered with a curtain wall or surfacing for aesthetics, and may include a drainage layer between the surface finish and the joints between shafts, or gravel columns behind the joints combined with horizontal relief holes through the joints. Some examples of drilled shaft walls were illustrated in Section 9.2.5.



Secant Wall Tangent Wall Soldier Pile Wall

Figure 9-32 Secant, Tangent, and Soldier Pile Walls

Drilled shaft walls are typically considered for applications when a cut is to be made and conditions require a wall that can be constructed in advance of the cut. The soldier pile walls may be constructed with steel sections and panels extending above the top of the shaft to support backfill above the top of the shaft. Reinforcement for secant and tangent piles can consist of a conventional reinforcement cage or structural steel sections. Drilled shaft walls can be used to support vertical loads as well as lateral earth pressures and water pressure.

Chapter 9 of GEC-9 describes a simple approach for design of walls constructed as a single row of drilled shafts. AASHTO Section 11.8 for nongravity cantilevered walls applies to the design of drilled shaft walls. The design of a drilled shaft foundation as a footing for a conventional gravity or semi-gravity cantilever wall may be accomplished using the concepts outlined in Section 9.3 of this chapter.

9.3.6 Design for Drilled Shaft Foundations with Lateral Movement of Soil Mass

Drilled shaft foundations may be subjected to lateral movement of a soil mass for some conditions such as a foundation subject to slope instability or lateral spreading during earthquake-induced liquefaction. In some cases, drilled shafts may be used with the intent to restrain the lateral movement of a soil mass as a remedial measure for an unstable slope. In these cases, the unstable soil mass can load the foundation with pressures as high as the passive earth pressure within the shifting soil mass. Drilled shafts designed to resist these lateral pressures must be embedded below the unstable soil mass to sufficient depth for geotechnical strength, and must be designed with sufficient structural strength to resist the stresses imposed by the unstable soil mass.

Lateral earth pressures above the potential sliding surface act on the drilled shaft, and the resultants from these pressures can be used to compute horizontal shear and overturning moment at the sliding surface. The drilled shaft below the sliding surface is then analyzed for geotechnical and structural strength and deformations using one of the methods described in previous sections. For groups of drilled shafts, possibly including an embedded cap, the resultant forces on the group are computed with the inclusion of group effects, as described in Chapter 11.

In general, the design of a structural foundation within an unstable slope would likely include remedial measures to improve overall stability of the slope. However, extreme event conditions such as liquefaction-induced lateral spreading may be a condition for which the foundation is designed to resist lateral forces from an overall stability failure. Note that liquefaction-induced lateral spreading may produce relatively low passive horizontal pressures within the liquefaction zone, but an intact overlying soil mass can contribute significant lateral pressures.

Lateral earth pressures can be as high as passive earth pressure for a drilled shaft within a large unstable soil mass. However, a lesser restraining force provided by a drilled shaft foundation may be sufficient to prevent a fully developed slope failure. Analyses of overall slope stability may be performed using a limit equilibrium approach such as the Modified Bishop, Simplified Janbu, or Spencer methods with an equivalent horizontal restraining force applied to the slope. In this case, the lateral force that must be provided by the foundation above the slip surface (and transferred to the soil or rock below the slip surface) would be the lesser of the passive lateral earth pressure or the restraining force necessary for overall stability of the sliding soil mass.

AASHTO (2017a) Section 11.6.2.3 requires that the evaluation of overall stability should be performed with a resistance factor of 0.65 in cases where the slope contains or supports a structural element.

Another approach to analysis of drilled shafts with lateral movement of a soil mass is to use the p-y method with a displacement offset of the p-y curves above the slip plane. In this manner, the analysis of shear resistance and bending moments in the shaft can be evaluated as a function of lateral soil movement above the slip plane. This approach can be particularly useful where drilled shafts are designed as a remedial slope stability measure and the slip plane is deep below the soil surface. The drilled shafts effectively act as shear dowels across the slip plane.

The results of the analyses shown in Figure 9-33 demonstrate the approach for a hypothetical slope failure composed of a deposit of clay overlying a rock formation, with the slip plane at the soil/rock interface. The behavior of the drilled shaft extending through the soil stratum into the rock is modeled by applying an offset of up to 3 inches to the p-y curves in the soil above the failure surface. The offset is applied with a transition to 0 inches offset over a thickness of 18 inches labeled in the figure as “shear zone”. As the soil movement increases, the shear and bending moment forces in the shaft increase as illustrated by

the shear and moment curves for different soil displacements in Figure 9-33. The curves shown include nonlinear soil vs. displacement relationships via the p-y curves, and nonlinear bending stiffness (moment-curvature) of the reinforced concrete drilled shaft as described in Section 9.3.3.2.

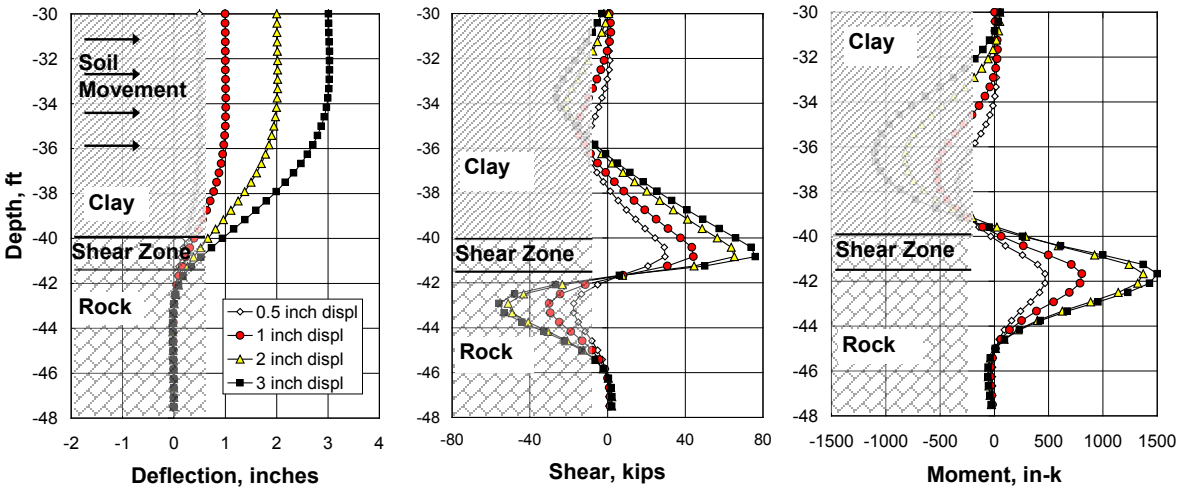


Figure 9-33 Analysis of Drilled Shafts in Moving Soil (Loehr and Brown, 2007)

This analysis approach provides a means to evaluate:

- Geotechnical strength of the socket below the shear plane. The shaft must extend to sufficient depth to provide the required shear force for overall stability.
- Geotechnical strength of the overall stability of the slope. The shear force provided by the drilled shaft can be applied as a restraining force for the limit equilibrium analysis of overall stability.
- Structural strength of the drilled shaft. The bending moments in the shaft associated with the shear force provide a means to evaluate structural strength at the conditions required for overall stability.
- Deformations of the slope. Estimates of slope movement associated with a remedial repair of a slope with marginal overall stability are likely to be approximate at best. The analysis provides a rational means of relating the restraining shear force to deformations of the slope.

As outlined above, a successful remedial design includes the restraining shear forces required to stabilize a slope mobilized as a shear force in the shaft by the shaft through the soil movement, and the maximum shear force which can be mobilized is at a magnitude which satisfies structural strength conditions for the shaft. Note that the p-y criteria in widespread use for laterally loaded drilled shafts are generally based on short term loading conditions; in clay soils, it is likely that some creep-induced movements will relax soil pressures and/or increase soil displacements with time. However, the analyses of shear and moment in the drilled shaft are not very sensitive to the p-y response of the soil for a deep failure surface.

Additional details of the use of analysis of slopes using the p-y method are provided by Loehr and Brown (2007) and in GEC-9.

Lateral loading due to liquefaction-induced instability in a soil mass from seismic loading presents a special problem and is discussed in detail in the following section.

9.4 DESIGN FOR SEISMIC LOADING

Design of bridges and other transportation structures to withstand the effects of earthquakes is a major challenge. The response of bridges to seismic ground motions is complex and involves time-varying nonlinear material behavior. Seismic ground motions are transmitted to the superstructure through kinematic interaction with the foundations, which could produce an excitation at the base of columns that differs from the free-field ground response. The superstructure mass is accelerated, generating inertial forces that must be transmitted back through the foundations and into the ground while the bridge continues to support non-seismic load components. Because seismic energy originates in the ground and the resulting forces must ultimately be transferred back into the ground, soil-foundation interaction plays a key role in the response of transportation structures to earthquakes.

A rigorous dynamic analysis of the complex interaction between the free-field ground motion, foundations, and the structure is not practical for most bridges, although it is warranted under some conditions, particularly for large nonconventional bridges. The approach of analyzing the complete system simultaneously including the ground and structure response is known as the "direct method" of seismic analysis. For routine design of conventional bridges, a more common approach is to divide the system into multiple components (*e.g.*, free-field ground response, foundation response, and structure response), simplify each to the desired level of complexity, solve independently, and recombine the output. This approach, which requires iteration to achieve compatibility between the individual analyses, is known as the "substructure method" as opposed to the direct method of analysis. The key components of this simplified approach to seismic design are:

1. Quantifying the response of the free-field ground to seismic excitation, for example by generating a response spectrum or acceleration time series to be used for structural analysis.
2. Structural analysis to quantify demand on individual bridge components, including foundations.
3. Foundation analysis in support of the seismic structural analysis, and foundation design to satisfy strength and deformation requirements.

The intensity of the seismic demand depends on proximity to seismic sources, regional and local ground conditions, and the design return period of interest. This step is typically performed in a probabilistic fashion and is known as "probabilistic seismic hazard analysis," and may be based on mapped values of spectral acceleration or more rigorous site-specific analyses. Although this task it often performed by the geotechnical engineer that is also responsible for foundation design, it is not covered in this manual. Procedures for defining response spectra are outlined in AASHTO (2017a), and a more detailed discussion is provided in FHWA-NHI-15-004 "LRFD Seismic Analysis and Design of Bridges" by Marsh et al. (2014).

Foundation designers are primarily responsible for item 3 from the above list; however, they play a key role in successful structural analysis performed by the structural designer for item 2. The foundation designer is responsible for quantifying foundation stiffness in terms of equivalent springs to be used in the structural analysis and must also inform the structural designer regarding substructure demands due to permanent seismic ground deformation. Seismic structural analysis and foundation analysis and design are concurrent efforts that require iteration and clear communication between foundation and structural designers. This section provides a summary of seismic structural analysis methods, followed by a description of seismic aspects of drilled shaft analysis and design.

9.4.1 Framework for Structural Analysis of Earthquake Effects

The focus of this manual is on the structural and geotechnical information needed to establish loads and resistances of drilled shafts under earthquake effects in accordance with current AASHTO LRFD Bridge Design Specifications (2017a), which utilize the force-based method (FBM) of seismic design. An alternative displacement-based method (DBM) is presented in the AASHTO Guide Specifications for LRFD Seismic Bridge Design (2015). In many regards, foundation analysis and design is performed in the same manner whether a FBM or DBM is used for the overall design of the structure. The objective of both approaches is to ensure that the structure has sufficient capacity to avoid collapse, with an implicit assumption that preventing collapse will prevent loss of life. Recognizing that constructing bridges in high-seismicity regions to remain elastic during major earthquakes would be prohibitively expensive, bridges are designed so that certain components in plastic-hinge zones will yield, particularly above-ground substructure elements such as columns or piers, but in a controlled manner to ensure that additional deformation can be accommodated without collapse. This behavior is known as a "ductile" inelastic response. By exhibiting ductile inelastic behavior, plastic hinges act as structural fuses to limit the forces transmitted to other bridge components such as foundations and connections. This approach is beneficial in that column damage can be readily assessed and repaired following a major earthquake, whereas damage to foundations is difficult and impractical to detect and repair. This is the motivation behind the Caltrans Type II column-shaft connection shown in Figure 9-2.

Structural analysis to quantify seismic demand for conventional bridges is generally performed in the same manner regardless of whether FBM or DBM will be used for design, typically using one of the following elastic methods:

- Equivalent-static methods wherein seismic demand is quantified from an acceleration response spectrum (for use with FBM) or displacement response spectrum (for use with DBM) at the fundamental period of the bridge. The fundamental period of the bridge is determined directly from the stiffness of the bridge in response to a uniform applied load, or from a modal analysis which considers the distribution of mass and stiffness based on energy principles. Single or multi-modal response can be considered in a response spectrum analysis. This is the most common approach for conventional bridges.
- Elastic response history analysis which computes the displacement and force history of components in response to an input ground motion using time stepping methods.

Nonlinear methods of analysis include the equivalent-static "pushover" method and nonlinear response history analysis. Nonlinear analysis methods require defining the stress-strain "backbone" curve or hysteretic response of bridge components and are most often used with the DBM. While informative, these methods are computationally expensive, require advanced user expertise, and can still involve significant uncertainty. The reader is referred to Marsh et al. (2014) for a complete description of elastic and nonlinear analysis methods.

The primary difference between FBM and DBM is how the results of the structural analysis (demands) are implemented in the design of individual bridge components, and how performance of the overall structure is assessed. In the FBM, design forces for yielding elements such as columns are computed from elastic structural analysis and then reduced by a response modification factor ("R-factor"), where the reduced force corresponds to the plastic capacity of the elements. It is therefore anticipated that the element will yield during the design earthquake since it is not designed to resist the full elastic demand. The remaining elements are then designed to remain elastic while resisting force effects transmitted through the plastic hinge zones, and therefore do not require costly detailing to achieve ductile behavior.

These remaining elements are said to be "capacity-protected" because the plastic hinge zones limit the imposed demand.

Foundations, including drilled shafts, are typically capacity-protected components and thus are designed to remain elastic during the design earthquake. When using the FBM, this corresponds to an R-value of 1.0, which is required for drilled shaft design by AASHTO (2017a) Article 3.10.9.4 for bridges in Seismic Zones 3 or 4, and recommended for critical or essential bridges in Seismic Zone 2. For ordinary bridges in Seismic Zone 2, Article 3.10.9.3 permits using one-half the R-factor for the substructure component type (*e.g.*, column) supported by the foundation, but R/2 cannot be taken as less than 1.0. This criterion permits limited foundation inelasticity in low seismicity regions. Furthermore, drilled shafts need not be designed for force effects exceeding those corresponding to column plastic hinging with consideration of material overstrength if these force effects are less than the elastic demand.

In the DBM, displacements rather than forces are extracted from the structural analysis output. Yielding elements in plastic-hinge zones are detailed so that the displacement corresponding to the full capacity of the element (*i.e.*, at failure) exceeds the computed displacement demand, where the ratio of the displacement capacity to demand is referred to as the component's displacement ductility factor μ. Minimum required values of μ are identified in the AASHTO Guide Specifications (2015) based on the component and bridge type and the level of seismic hazard. As in the FBM, remaining bridge components are capacity-protected and are designed for elastic demands.

Whereas ductility is considered explicitly in the DBM, prescriptive detailing procedures are utilized in the FBM to achieve a target ductility without explicitly considering the inelastic behavior of the elements. As a result, there is more uncertainty about the actual behavior of a bridge designed using the FBM, particularly with regard to maximum deformations. For this reason, nonconventional bridges (*e.g.*, cable-stayed bridges) and other lifeline transportation structures are often designed using the DBM. DBM are particularly useful when designing to higher performance criteria than collapse-prevention, which are usually defined in terms of maximum permissible displacements or rotations of critical components.

Some owners permit formation of below-ground plastic hinges and limited inelasticity in drilled shafts, typically in conjunction with the DBM in accordance with the AASHTO Guide Specifications (2015). This approach is most often used with extended pile-shafts or in cases of liquefaction-induced lateral spreading for which preventing plastic hinge formation below ground is prohibitively expensive.

It is good practice for the foundation designer to communicate with the structural designer regarding (1) whether a FBM or DBM is being performed, and therefore which version of the AASHTO code applies, and (2) whether below-ground plastic hinge formation is part of the strategy for earthquake resistance.

Regardless of the method of seismic structural analysis, the structural designer is responsible for determining the factored lateral force effects (shear, moment, and axial load demand) transmitted to drilled shaft foundations, which are then designed to satisfy the LRFD criterion. The p-y method previously presented in this chapter is well-suited for analyzing the response of drilled shafts to seismic force effects from the superstructure while taking the following seismic issues into account:

- Changes in soil behavior due to seismic excitation, including excess pore water pressure generation and liquefaction of granular soils or cyclic softening potential of cohesive soils, and
- Potential for permanent lateral ground displacement resulting from loss of soil strength, which can impart large forces to the bridge through its foundations.

These issues are discussed in the following sections.

9.4.2 Effects of Excess Pore Water Pressure Generation including Liquefaction

In saturated, loose cohesionless soils, cyclic shear strain during ground shaking generates excess pore water pressure, reducing effective stress. Because the strength and stiffness of cohesionless soil is proportional to effective stress, a reduction in effective stress due to seismic excitation results in a decrease in soil lateral resistance, which must be considered for cohesionless layers for the Extreme Event 1 limit state. Similarly, a decrease in effective stress results in a decrease in axial side resistance, which is discussed in Section 10.7.

The magnitude of pore water pressure in excess of the pre-earthquake static condition, Δu, is often expressed as a fraction of the pre-earthquake vertical effective stress σ'_{v0}, known as the excess pore water pressure ratio r_u:

$$r_u = \frac{\Delta u}{\sigma'_{v0}} \qquad 9\text{-}12$$

If the excess pore water pressure approaches the pre-shaking vertical effective stress (*i.e.*, $r_u \rightarrow 1$), effective stress is reduced to essentially zero and liquefaction occurs. Liquefaction can drastically reduce the resistance of drilled shafts to lateral and axial loads and may cause secondary effects such as lateral spreading that impose additional loads on the bridge substructure. While the focus of this section is on liquefaction, generation of significant excess pore water pressure even in the absence of liquefaction can have significant consequences and should be considered during foundation design as discussed below.

Idriss and Boulanger (2008) present a three-part framework for evaluating liquefaction consisting of successive steps to analyze the susceptibility, triggering potential, and consequences of liquefaction. The reader is referred to this reference as well as Kavazanjian et al. (2011) and FHWA-NHI-15-004 "LRFD Seismic Analysis and Design of Bridges" by Marsh et al. (2014) for more detailed information and guidance.

9.4.2.1 Susceptibility

Soils most susceptible to liquefaction are loose, non-plastic or low-plasticity, recently deposited cohesionless sediments including natural soil deposits and made-made fills, especially hydraulically-placed fill. In general this precludes cohesive soils and granular soil with a moderate to high fines content, particularly if the fines exhibit plasticity. Susceptibility is assessed on the basis of grain size distribution, Atterberg limits, and/or CPT soil behavior type index I_c (Idriss and Boulanger 2008). Susceptibility to liquefaction should always be assessed prior to performing liquefaction triggering analysis, and soil layers found not susceptible to liquefaction should be excluded from the triggering analysis. This screening procedure is necessary to produce reliable liquefaction triggering predictions, since methods for quantifying resistance to liquefaction triggering are not inherently formulated to account for susceptible versus non-susceptible soils and therefore may produce a misleading prediction of liquefaction in non-susceptible layers.

9.4.2.2 Triggering

Triggering refers to the onset of liquefaction and for computational purposes corresponds to $r_u = 1.0$. Liquefaction triggering analyses are most-often performed using the "simplified method" (Seed and

Idriss, 1971) in which seismic demand at a given depth is quantified in terms of maximum cyclic shear stress, and resistance to liquefaction is computed on the basis of penetration resistance (CPT or SPT). These parameters are normalized by effective overburden pressure and referred to as cyclic stress ratio (CSR) and cyclic resistance ratio (CRR), respectively, and the factor of safety against liquefaction is interpreted as the ratio FS_{liq} = CRR/CSR. CPT-based methods are generally preferred over SPT because the continuous profile of measurements is better suited to defining stratigraphy in cases of interbedded loose and dense soil layers common in alluvial environments susceptible to liquefaction. CPT data can also easily be imported into liquefaction prediction software that is now commonplace.

Empirical relationships between penetration resistance and liquefaction resistance are based on evaluating case history databases of the occurrence or absence of surface manifestation of liquefaction at sites subjected to earthquakes, and are updated frequently. Users should consult the original reference for a chosen triggering prediction method to ensure it encompasses the conditions at the project site of interest, and to understand the limitations of each method. Since the publication of the previous version of this manual in 2010, many case histories have been added to liquefaction triggering databases at sites affected by large earthquakes including the 2010-2011 Canterbury earthquake sequence in New Zealand and the 2011 Tohoku earthquake in Japan. These case histories have shed new light on several aspects of liquefaction triggering of interest to deep foundation design, including the influence of fines content, shaking duration, and large magnitude earthquakes.

The result of a liquefaction triggering analysis is a profile of FS_{liq}. This profile is used to delineate layers predicted to undergo liquefaction ($FS_{liq} \leq 1.0$), layers that are not predicted to undergo liquefaction but which may generate significant excess pore water pressure ($1.0 < FS_{liq} \lesssim 1.5$), and remaining layers that are not susceptible to strength loss. Layers found susceptible to cyclic softening as described subsequently would also be delineated. Liquefaction susceptibility and triggering analysis should be performed early in the drilled shaft design process so that trial designs can be selected with sufficient embedment below soil layers predicted to liquefy.

9.4.2.3 Consequences

Values of FS_{liq} along with penetration resistance and index properties are then used to estimate parameters necessary to assess the consequences of liquefaction. The important parameters include post-liquefaction residual undrained strength $s_{u,r}$, maximum cyclic shear strain γ_{max}, and r_u (for cases of $1.0 < FS_{liq} \lesssim 1.5$) Commonly-used software programs for liquefaction susceptibility and triggering analysis will generate profiles of these parameters automatically.

The primary consequence of liquefaction that affects drilled shafts is a reduction in axial and lateral resistance, both in terms of stiffness (load versus deformation behavior) and strength (nominal resistance). Reductions of drilled shaft lateral stiffness can change the distribution of forces in the shaft, often driving the maximum flexural demand deeper and amplifying its magnitude. The reduction in stiffness also modifies the dynamic response of the superstructure, which can further modify the inertial force effects transmitted to the foundations. Methods for incorporating these effects into drilled shaft lateral loading analyses are described below. Axial effects are described in Section 10.7, and effects on pile groups are discussed in Chapter 11.

An important aspect of liquefaction evaluation is to consider that the consequences are generally proportional to the severity of liquefaction (*e.g.*, as quantified by FS_{liq}) and depend on the thickness, depth, and continuity of liquefied layers relative to the drilled shafts, the presence or absence of a slope and the gradient of the slope relative to the depth of liquefied layer(s), duration of ground shaking, and

other factors including the stratigraphy above and below the liquefied layer. The influence of many of these factors is not apparent based on values of FS_{liq} alone, and requires further interpretation and judgment. For example, a thin soil layer that is marginally susceptible to liquefaction located beneath a thick stratum of nonliquefiable soil would likely have negligible consequences with respect to bridge performance. Likewise, Figure 9-34 shows that although medium dense and dense cohesionless soil could reach liquefaction triggering (r_u = 1.0), the potential for significant shear strain to develop is limited, which in turn limits the potential for detrimental vertical and/or horizontal soil deformation (Ishihara and Yoshimine, 1992; Idriss and Boulanger, 2008; Cetin et al., 2009). Thus, the consequences of liquefaction with regards to drilled shaft performance should be quantified explicitly on the basis of parameters that capture the potential for strength loss (such as r_u or $s_{u,r}$) or soil deformation (γ_{max}) rather than on the basis of a binary liquefaction versus no-liquefaction criterion.

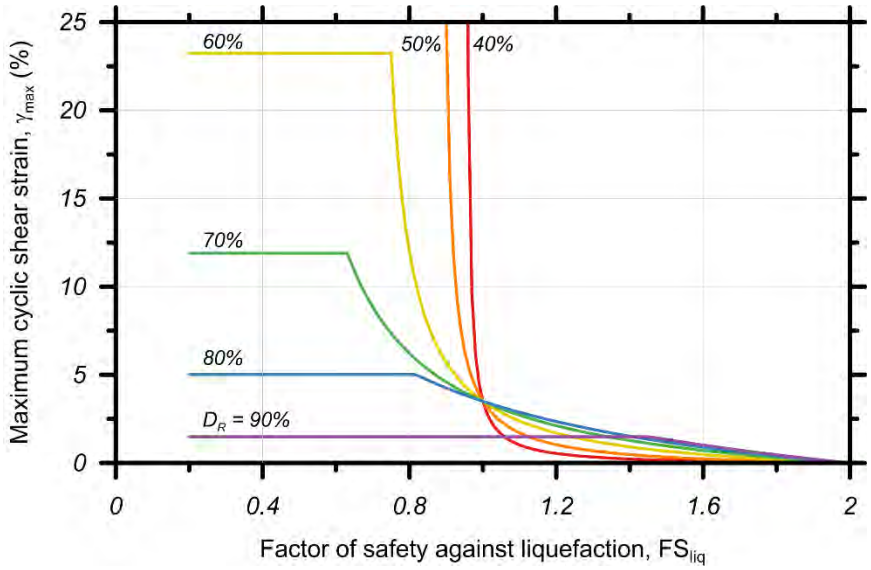


Figure 9-34 Maximum cyclic shear strain as a function of factor of safety against liquefaction and relative density after data from Ishihara and Yoshimine (1992) based on equations presented in Idriss and Boulanger (2008).

Residual strength of liquefied soil can be estimated on the basis of CPT or SPT penetration resistance and σ'_{v0}. The median relationship between corrected SPT resistance and $s_{u,r}$ by Kramer and Wang (2015) is shown in Figure 9-35 and given by Equation 9-13:

$$s_{u,r} = \exp\left(-8.444 + 0.109\left[(N_1)_{60}\right] + 5.379\left(\sigma'_{v0}\right)^{0.1}\right) \geq \textit{drained strength} \qquad 9\text{-}13$$

Where:

$(N_1)_{60}$ = Overburden- and energy-corrected SPT blow count
σ'_{v0} = Pre-earthquake vertical effective stress in atmospheres
$s_{u,r}$ = Estimated residual undrained strength in atmospheres

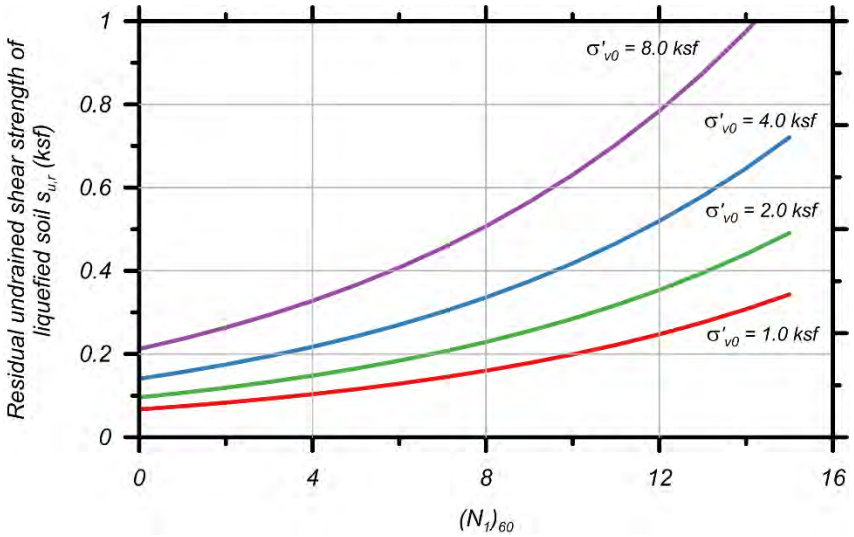


Figure 9-35 Relationship between corrected SPT resistance and post-liquefaction residual undrained strength after Kramer and Wang (2015).

Additional correlations between $s_{u,r}$ and SPT/CPT penetration resistance are summarized in Idriss and Boulanger (2008).

The magnitude of maximum shear strain γ_{max} can be estimated based on the computed FS_{liq} and relative density of the soil using procedures given by Ishihara and Yoshimine (1992) and approximated by Idriss and Boulanger (2008) with a series of equations that are plotted for different values of D_R in Figure 9-34. The behavior illustrated in Figure 9-34 shows that soils with a high relative density do not develop significant shear strain even if liquefaction is triggered. Idriss and Boulanger (2008) provide equations describing the curves in Figure 9-34 for spreadsheet implementation. Use of these equations/Figure 9-34 should be limited to soils with $D_R \geq 40\%$; for looser soils significant shear strain potential is expected immediately upon liquefaction triggering (refer to the trends in Figure 9-34). Equivalent relationships based on SPT or CPT resistance in place of D_R are also given in Idriss and Boulanger (2008).

Methods to account for excess pore water pressure generation in soil layers with $1 < FS_{liq} \lesssim 1.5$ require estimation of r_u. The simplest approach is a stress-based framework that relates FS_{liq} to r_u directly as presented by Marcuson et al. (1990). The range of data presented by Marcuson et al. can be approximated by the following hyperbolic best-fit:

$$r_u = \frac{1}{1+15\left(FS_{liq}-1.0\right)} \quad for \quad FS_{liq} > 1 \qquad 9\text{-}14$$

Research has demonstrated that shear strain is more closely associated with pore water pressure generation and liquefaction triggering than shear stress. An alternative approach to Equation 9-14 is to estimate r_u as a function of maximum cyclic shear strain using the following relationship developed by Cetin and Bilge (2012):

$$r_u = \exp\left\{\ln\left[1-\exp\left(-\frac{-0.407\gamma_{\max}}{-0.486+0.025\ln\sigma'_{v0}-\frac{D_R}{100}}\left(\frac{0.620}{1+\gamma_{\max}}+1\right)\right)\right]\right\} \qquad 9\text{-}15$$

Maximum cyclic shear strain for use in Equation 9-15 can be determined from the equations given in Idriss and Boulanger (2008) and plotted in Figure 9-34 based on FS_{liq}.

9.4.2.4 Liquefied Soil in p-y Analyses

Multiple approaches are available for incorporating the effects of liquefaction into p-y curves including:

- Using a soft clay p-y model to represent the undrained behavior of liquefied soil, with the ultimate resistance p_{ult} set to $s_{u,r}$ (e.g., Wang and Reese, 1998).
- Using a p-y model for sand with a p-multiplier to account for reductions in p-y stiffness and p_{ult} (*e.g.*, Brandenberg et al., 2007).
- Using a p-y curve specifically formulated for liquefied sand such as Franke and Rollins (2013).

The approach of using a soft clay p-y curve with p_{ult} based on $s_{u,r}$ is self-explanatory and simple to implement. Wang and Reese (1998) recommend using $\varepsilon_{50} = 0.05$ when defining the Matlock (1970) soft clay p-y curve to represent liquefied soil. The cyclic reduction factor option in Matlock's soft clay p-y formulation is meant to account for degradation of sensitive clays when subjected to many loading cycles from wind or wave action and is generally not applicable to seismic problems (Brandenberg et al., 2007).

Figure 9-36 presents relationships for estimating p-multipliers (P_m) to represent liquefaction based on results of centrifuge model studies (Brandenberg, 2005) and back analysis of case histories (AIJ, 2001). The relationships were developed by adjusting p-multipliers for liquefied layers until the computed results provided a good match to measured pile behavior in terms of bending moment and/or deflection. Typical p-multiplier values range from about 0.05 to 0.25 for loose to medium-dense sands susceptible to liquefaction. The result of this approach is a p-y curve with similar overall shape as nonliquefied sand, but with reduced ultimate resistance and stiffness as shown in Figure 9-37. P-multipliers are traditionally used to represent shadowing effects for pile groups as discussed in Chapter 11; group-reduction p-multipliers can be combined with the effects of liquefaction described here.

A similar approach is to model reduced soil unit weight to simulate the effect of reduced effective stress due to excess pore water pressure generation. Brown and Camp (2002) obtained good agreement with the results of a lateral load test of an 8-ft diameter drilled shaft in loose to medium dense silty sand subjected to blast-induced liquefaction by using unit weights equivalent to 15% of the soil unit weight. The effect is similar to that developed using a p-multiplier of a similar value. However, modeling reduced soil unit weight affects the vertical effective stress for underlying layers, so this approach should be used with caution if liquefaction is not confined to thin surficial layers.

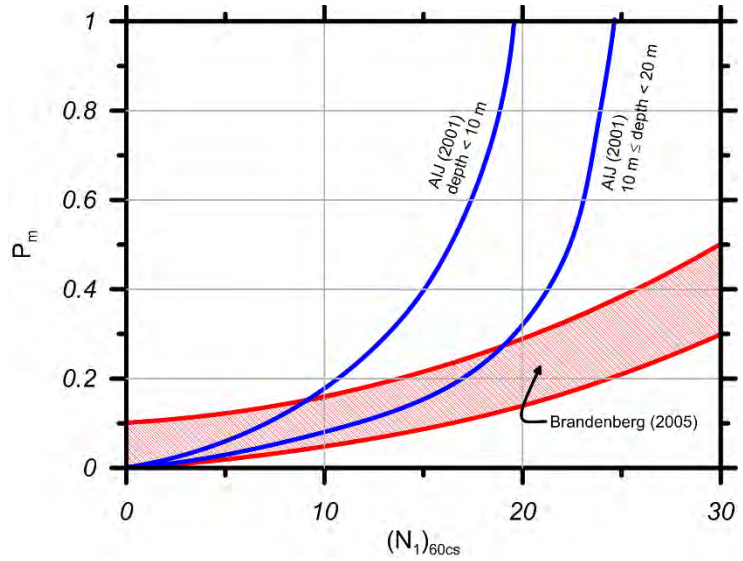


Figure 9-36 P-multipliers for representing liquefied soil based on clean-sand corrected SPT blow count from various studies (after Brandenberg et al., 2007).

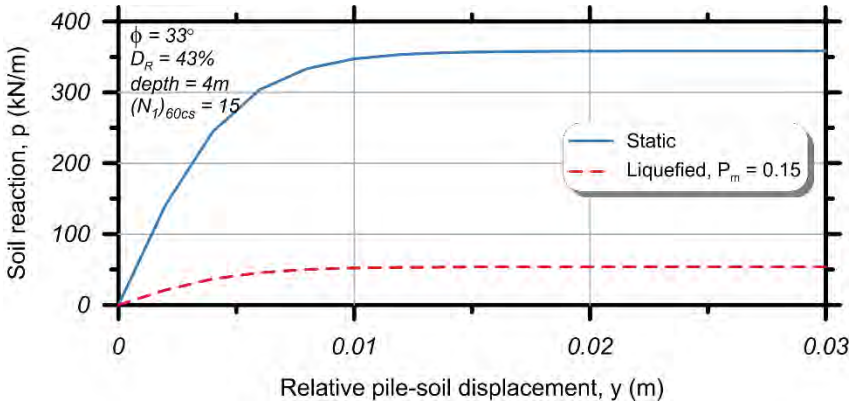


Figure 9-37 Example of API (1993) p-y curve for sand modified to account for liquefaction using p-multiplier approach.

Rollins et al. (2005) measured the response of large-scale pile groups subjected to blast-induced liquefaction. While the soil was liquefied, lateral loads were applied to simulate inertial seismic loading. In this context, "inertial" lateral loads are those generated by oscillation of the superstructure and imposed at the pile head, whereas "kinematic loads" are generated by permanent ground displacement such as lateral spreading. P-y curves interpreted from the Rollins et al. (2005) experiments exhibit a concave-up shape at large displacement because of shear-induced dilation of the liquefied soil when subjected to continuous straining as the piles were pushed through the soil. Kashighandi and Brandenberg (2010) and Ashford et al. (2011) suggest that the dilative response of liquefied soil at large relative displacement (y) should not be incorporated into the equivalent-static analysis method for kinematic lateral spreading loads. This is because temporary dilation is associated with periods of lower ground acceleration and corresponding drops in pore water pressure, during which ground displacement relative to deep foundations is reduced and significant soil-pile load transfer does not occur. These time-dependent effects are not captured in an equivalent-static analysis, leading to the recommendation that a concave-down p-y curve be used to represent liquefied soil for kinematic loading. Concave-up curves can also present numerical convergence problems for lateral pile analysis software.

Franke and Rollins (2013) proposed a hybrid p-y curve that is a combination of the Rollins et al. (2005) concave-up curve and the Matlock soft clay curve with p_{ult} based on $s_{u,r}$, where the value of p at a given displacement is taken as the lesser of the two formulations. This limits the ultimate resistance at large strain to a value consistent with $s_{u,r}$. Franke and Rollins (2013) demonstrate that the hybrid model provides good agreement with measured or observed behavior for back-analysis of cases of inertial and kinematic loading.

For cases where lateral spreading displacement is significant, the shape of the p-y curve is not a controlling parameter and any of the above approaches are likely to yield similar results. Maximum drilled shaft flexural demands and displacement are much more sensitive to the properties of nonliquefied crust soil overlying the liquefied layers, since the crust imposes larger loads.

For soils in which significant excess pore water pressure is generated but liquefaction triggering is not reached (*i.e.*, $1 < FS_{liq} \lesssim 1.5$), a reduction in lateral resistance in proportion to the reduction in effective stress is expected; however, the exact relationship between r_u and lateral resistance is not well understood. Limited research has been conducted to quantify the effect of $0 < r_u < 1$ on p-y curves. Chang and Hutchinson (2013) performed a series of large-scale shake table tests on a steel pile in a soil-filled laminar container in which the soil was liquefied by shaking the base of the container, followed by application of a lateral load at the pile head. During pile loading, values of r_u over the length of the pile ranged from about 1% to 20%. The p-y response interpreted from the test measurements exhibited a concave-up shape for soil near the surface, similar to the behavior documented by Rollins et al. (2005). Values of r_u in these upper soil layers ranged from about 10-20%. For soil near the bottom of the pile where r_u was less than 10% during pile loading, the curves resembled the static condition and did not exhibit a concave-up shape. Chang and Hutchinson recommend scaling factors to apply to the Rollins et al. (2005) concave-up curve for cases of $0.1 < r_u < 1.0$, which can be used for inertial loading.

In the absence of more comprehensive research on this subject, p-multipliers for the condition $1 < FS_{liq} \lesssim 1.5$ can be estimated by linearly interpolating between values corresponding to r_u = 100% (*e.g.*, determined from Figure 9-36; termed $P_{m,liq}$ in the following equation), and the estimated r_u (from Equation 9-14 or 9-15) as follows (after Dobry et al., 1995):

$$P_m = 1 - r_u\left(1 - P_{m,liq}\right) \quad for \quad 1 < FS_{liq} \le 1.5 \qquad 9\text{-}16$$

For soil layers with $FS_{liq} \gtrsim 1.5$, significant excess pore water pressure generation is not anticipated, and stiffness or strength degradation does not need to be incorporated into the p-y curves. The p-y curves for these layers may be adjusted to account for dynamic effects other than excess pore water pressure generation, as discussed subsequently.

9.4.2.5 Liquefaction-Induced Lateral Spreading or Flow Failure

When soil underlying a slope or in close proximity to a free-face such as a river bank undergoes liquefaction, the presence of a static driving shear stress from the slope (τ_{static}) combined with inertia due to ground shaking can result in the gradual accumulation of displacement of near-surface liquefied soil and overlying nonliquefied "crust" in the downslope direction. This mechanism is defined as lateral spreading and applies to cases where the shear strength of the liquefied soil exceeds τ_{static}, such that displacement only occurs when an additional inertial driving force acts in the downslope direction. By definition, lateral spreading displacement therefore occurs simultaneously with seismic inertial forces.

Lateral spreading is one of the most damaging consequences of liquefaction. For cases in which $s_{u,r} < \tau_{static}$, the ground is statically unstable even in the absence of inertial forces caused by shaking, which typically leads to large, unbounded permanent displacement on the order of tens or hundreds of feet. This latter mechanism is known as a flow slide. Both mechanisms can result in large kinematic loads imposed on drilled shafts, particularly when a nonliquefied crust layer is present. When a flow slide is anticipated, it is usually necessary to mitigate liquefaction with ground improvement such that the flow slide is prevented, because the extent of ground failure would otherwise render the surrounding infrastructure unusable even if individual bridge bent foundations could resist the imposed loads.

Methods for designing bridges to resist kinematic lateral spreading loads are summarized in Kavazanjian et al. (2011) based on the procedure described in the MCEER/ATC-49-1 Liquefaction Study Report (2003), and discussed in further detail in Ashford et al. (2011). The procedures described in these references consist of equivalent-static analysis methods in which a profile of free-field lateral spreading displacement is imposed on the free ends of p-y springs to simulate kinematic loading. Step-by-step guidance for performing equivalent-static lateral spreading analysis is provided in these references and will not be repeated here. The equivalent-static analysis method has been demonstrated to provide a good match to well-documented case histories of various levels of bridge performance including examples by Brandenberg et al. (2013), Ledezma and Gonzalez (2014), Turner et al. (2016), and Franke and Rollins (2017).

Nonlinear dynamic continuum analyses are sometimes performed for critical structures subjected to lateral spreading. These analyses involve considerable uncertainties and are not amenable to parametric evaluation because they are time consuming, and thus are not suitable for routine design of conventional bridges. Their use may be warranted for verifying equivalent-static analyses of critical lifeline bridges.

Figure 9-38 illustrates the basic components of the equivalent-static analysis method for lateral spreading:

- Geotechnical properties and results of liquefaction triggering analysis are used to define p-y springs for liquefied and nonliquefied soil layers using the methods discussed above.
- A profile of estimated lateral spreading displacement versus depth is imposed on the free ends of the p-y springs.
- Foundation(s) and other substructure components that mobilize resistance to lateral spreading, or experience a demand due to lateral spreading (*e.g.*, columns, foundation caps, column-to-deck connections, etc.) are represented with structural elements or springs in the model.
- The analysis is executed to determine the substructure response.

Methods for estimating free-field lateral spreading ground displacement (e.g., Youd et al., 2002; Zhang et al., 2004; Faris et al., 2006) produce highly variable results, reflecting the complexity of lateral spreading behavior and a large degree of epistemic uncertainty. However, if significant lateral spreading deformation on the order of several feet or more is expected, the response of drilled shafts is unlikely to be sensitive to the exact level of ground displacement because passive pressure of nonliquefied crust layers will have been fully mobilized. For cases where multiple lateral spreading displacement prediction methods are in agreement that displacement will be minimal, the foundation designer may choose to design for the low level of displacement. For cases where significant displacement is expected, or where there is a lack of agreement between multiple prediction models, the foundation design should be stable against fully-mobilized nonliquefied crust loads. Equivalent-static p-y methods of analysis provide a means for rapidly evaluating a trial drilled shaft design and assessing multiple group and size configurations. Caution should also be exercised so as not to blindly apply the lateral spreading prediction methods to conditions outside the empirical range from which they were formulated. For

example, liquefaction at significant depth or at a large distance from a free face is unlikely to result in lateral spreading because τ_{static} is minimal in these zones.

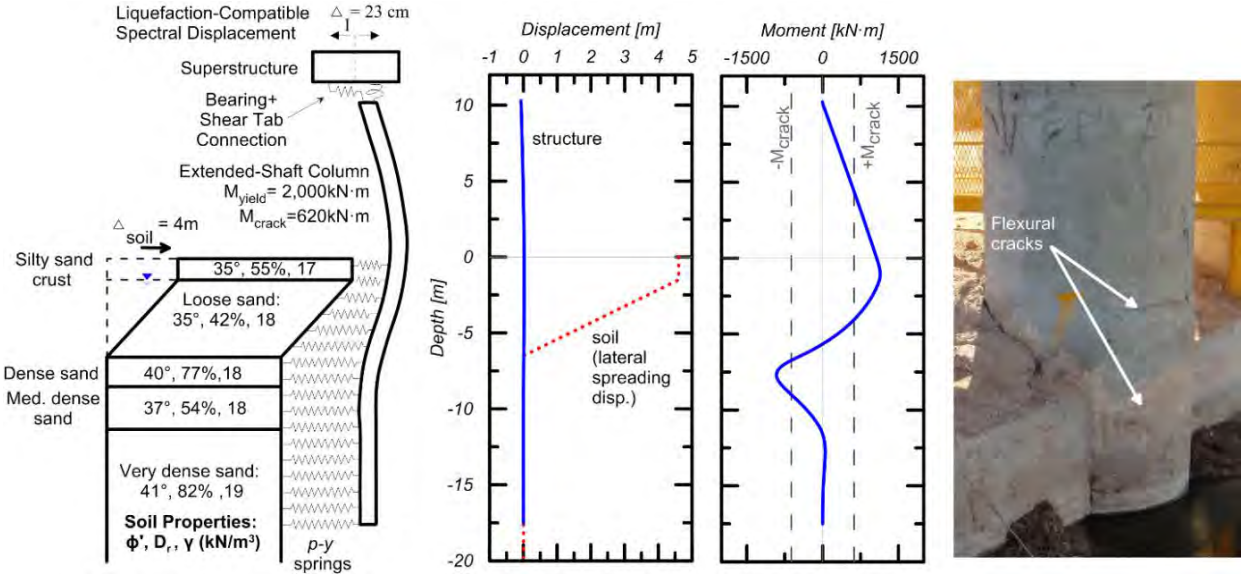


Figure 9-38 Equivalent-static analysis of San Felipito Bridge in Baja, Mexico, subjected to lateral spreading during 2010 El Mayor Cucapah earthquake provides good match to observed flexural cracking but low residual column displacement shown in photo at right (after Turner et al., 2016). Photo courtesy B. Turner.

In most cases, the appropriate soil displacement input for equivalent-static lateral spreading analysis is the free-field ground displacement. When the concept of "pile pinning" as originally presented in MCEER/ATC-49 and Boulanger et al. (2005) is applied to approach embankments over liquefied soil exerting kinematic demands on abutments, it is necessary to quantify the (finite) out-of-plane width of the embankment relative to the abutment pile group for implementation in a two-dimensional analysis. For interior bridge bents subjected to a broad field of laterally spreading ground, a reduction in demand does not occur due to pile pinning unless the passive soil wedge that forms upslope of the bent extends beyond the limits of lateral spreading (Turner and Brandenberg, 2015). Lateral spreading displacement upslope of stiff deep foundations is often observed to be less than the adjacent free-field displacement during post-earthquake observations. The reduced displacement is evidence that kinematic load transfer has occurred between the ground and foundations, which is the mechanism captured by the equivalent-static p-y method of analysis.

While lateral spreading, by definition, takes places concurrently with inertial loading, maximum superstructure inertial loads in response to the largest amplitude cycles of ground shaking, which usually occur near the beginning of an earthquake, are unlikely to occur at the same time as maximum kinematic loads. Kinematic load transfer between laterally spreading soil and deep foundations generally increases in proportion to total accumulated soil displacement, but also depends on transient changes in soil stiffness resulting from fluctuations in excess pore water pressure during and between successive cycles of shaking (Brandenberg et al., 2007). Ashford et al. (2011) provide reduction factors to modify inertial demands that account for the fraction of maximum inertia that occurs concurrently with kinematic demands, as well as a factor that accounts for the influence of liquefaction on spectral acceleration. These factors are experimentally validated and provide a means to represent complex dynamic behavior in an equivalent-static analysis framework. The factors vary depending on spectral shape to approximately capture the influence of ground motion characteristics and intensity in different seismic regimes. Some

state DOTs specify a reduction factor to be applied to inertial loads, including a factor of 0.5 in California (Caltrans, 2013) and 0.25 in Washington State (WASHDOT, 2015). Regardless of the reduction factor applied to inertial loads, the force effects imposed on the drilled shaft for design should not exceed the force corresponding to column plastic hinge formation if the columns are designed to be a ductile earthquake resisting element.

Equivalent-static lateral spreading analyses are often performed using a foundation(s)-only model that does not include above-ground substructure components such as columns or top-of-column connections. For inertial and non-seismic force effects that are generated in the superstructure and transmitted to the foundations, decoupling the foundation analysis in this manner is adequate because it follows the load path. However, such an approach is inadequate for capturing the reversal in load path direction that occurs during lateral spreading. Demands imposed on drilled shafts by lateral spreading result in top-of-foundation force effects (or equivalently, displacement and rotation) that are transmitted into the columns, thus affecting the top-of-column connections and the superstructure. These demands on above-ground substructure components are an outcome of the lateral spreading demand and therefore cannot be determined using a decoupled foundation(s)-only model in which the base-of-column force effects must be specified as boundary conditions prior to executing the analysis. Failing to adequately capture this behavior by using a decoupled foundation(s)-only analysis has two major implications:

- Lateral spreading demands on above-ground substructure components are not recognized or quantified and therefore are not communicated to the structural designer. This could lead to failure of the columns even if they were adequately designed for superstructure inertial loads. Failure of bridge columns or deck collapse due to excessive column deformation in response to kinematic lateral spreading loads has been documented in several case histories.
- If above-ground substructure components are adequately designed for kinematic demands, a potentially beneficial mechanism can be realized in which the loads are distributed through the superstructure to bents or abutments located outside the zone of lateral spreading, thereby lessening the demand on foundations in the lateral spread zone.

Case history observations including Cubrinovski et al. (2014), Turner et al. (2016), and Franke and Rollins (2017) have demonstrated that above-ground bridge components play an important role in the response of bridges to lateral spreading. To capture this behavior, above-ground substructure components must be represented in the equivalent-static analysis model, for example as illustrated in Figure 9-5. Software that performs decoupled foundation(s)-only analyses may be poorly suited for this task because of limited ability to model above-ground structural components or realistically implement boundary conditions representative of top-of-column connections. In this case, the foundation designer must use an alternative analysis tool and/or work closely with the structural designer to develop an analysis strategy that captures kinematic and inertial demands on the entire substructure.

9.4.3 Cyclic Softening of Cohesive Soils

Cohesive soils under seismically-induced cyclic loading can also undergo strength decreases, with accompanying decreases in drilled shaft resistance to lateral loads. This process is described as cyclic softening to distinguish it from liquefaction (Idriss and Boulanger, 2008). Soils most susceptible to cyclic softening are sensitive clays with low OCR. The effect of cyclic softening can be incorporated in lateral loading analyses by using a reduced value of undrained shear strength, s_u, when defining p-y curves. This approach inherently modifies both the stiffness and ultimate resistance of the p-y curve. A value equal to 80 percent of the static value of s_u provides a reasonable approximation to account for cyclic softening. More explicit procedures for estimating cyclic softening potential are presented in Idriss and Boulanger (2008), ranging from simplified methods using soil index properties to detailed analyses involving cyclic laboratory testing of undisturbed samples, which would not typically be warranted except for major structures in critical seismic zones. The method most readily implemented for routine projects quantifies the cyclic resistance ratio (CRR) of cohesive soils so that it can be compared to seismic demand (CSR) to quantify a factor of safety against cyclic softening within the same calculation framework for performing liquefaction triggering analysis. This method can be implemented alongside liquefaction susceptibility and triggering calculations and is included in commercial software packages.

9.4.4 Foundation Stiffness, Damping and Input Motions for Structural Analysis

Modeling of the support conditions at the connection between drilled shafts and the supported bridge typically takes one of two forms, both of which require the input of foundation designers. One approach is to establish a "depth of fixity" as discussed previously and model the bridge columns as being fixed at this depth. This approach is primarily intended to capture maximum moment demands and is considered adequate for bridges in AASHTO (2017a) Seismic Zones 2 and 3, for sites classified as A, B, C or D. The alternative approach is to model the foundation support through a set of springs to represent each degree of freedom. Structural designers often refer to these as "soil springs" although the term is misleading because the correct formulation includes the combined response of the drilled shafts and geomaterials through soil-foundation interaction, not the soil alone. This method requires input from foundation designers to select appropriate values of spring stiffness, most commonly based on p-y analysis to define lateral and rotational spring coefficients and t-z analysis for vertical spring coefficients. While elastic analytical solutions of foundation stiffness are available, they are generally inadequate for drilled shaft design in seismic regions because of layered geomaterial profiles, significant material nonlinearity, and the potential for degradation in weak soil layers, all of which can be captured with p-y and t-z methods. Use of springs to model foundation lateral response is strongly recommended for bridges in Seismic Zone 4, for sites classified as E or F in Seismic Zones 2 and 3, and can be applied to any site. Further guidance on defining foundation stiffness springs is provided in Kavazanjian et al. (2011).

Most structural analysis software requires that foundation springs be defined in terms of equivalent-linear stiffness values (*i.e.*, spring constants) as opposed to nonlinear curves. Because the lateral load versus deformation behavior of drilled shafts is strongly nonlinear, this requires close coordination between structural and foundation designers to ensure that the springs implemented in the structural model are consistent with the magnitude of force effects that will be imposed on the drilled shafts.

Commonly used p-y analysis method software will automatically generate stiffness matrices for a defined combination of force effects. The user should carefully select the options for computing the stiffness matrix to ensure compatibility with the structural analysis software, and should perform checks to ensure the reported stiffness values are consistent with the specified value of each force effect divided by the

computed deformation. Cross-coupling effects between degrees of freedom can also be significant and should either be included in a lumped foundation stiffness matrix with off-diagonal terms equal to zero, or explicitly defined if the structural analysis program can accept fully-populated stiffness matrices. An example cross-coupling effect is that a moment applied to a drilled shaft in the bridge longitudinal direction causes a displacement in the bridge longitudinal direction (in addition to rotation); a longitudinal shear also produces a displacement in the same direction. The longitudinal shear versus displacement stiffness is therefore dependent on the magnitude of the moment, and vice versa.

For use in equivalent-static structural analysis methods, including modal analysis, only foundation stiffness is required. For dynamic structural analysis, foundation damping can also be included through rate-dependent (viscous) dashpots or specified as an equivalent percentage of critical damping. Damping coefficients for each foundation degree of freedom can be computed from simplified analytical solutions (e.g., NIST 2012) or on the basis of dynamic load testing. For example, Brown (2007) presents a method for quantifying damping from the results of rapid lateral load tests by representing the system as an equivalent single degree of freedom model that accounts for the inertial effects of foundation mass, system damping, and static foundation stiffness. Alternatively, increases in dynamic resistance due to rate-of-loading and damping effects can be approximated by increasing p-y and t-z resistance when computing foundation spring stiffness. Paikowsky et al. (2004) analyzed a database of load test results for deep foundations axially loaded using the Statnamic® rapid test method versus static load testing and found that, on average, piles in clay exhibited a 54% increase in dynamic resistance compared to the equivalent static case, piles in silt exhibited a 45% increase, and piles in sand exhibited a 10% increase. For layers not susceptible to strength loss, as discussed in the previous sections, these values could be used to approximate drilled shaft response to dynamic lateral loads for preliminary design but should be verified with rapid load testing.

The final component of soil-foundation interaction that may be implemented in some designs is to quantify how top-of-foundation motions that excite the bridge superstructure differ from free-field motions due to kinematic interaction effects. (Note, this aspect of kinematic interaction is distinct from loads imposed by permanent ground failure such as lateral spreading.) Because the stiffness of drilled shafts differs from the surrounding geomaterials, the shafts do not necessarily conform to the profile of free-field ground deformation, leading to a difference in displacement between the ground surface and top-of-foundation. This effect is most pronounced for large-diameter drilled shafts at sites with soft near-surface soils, such as major bridges crossing soft alluvial or estuarine deposits. In such cases, the drilled shaft motion that excites the superstructure can be significantly decreased relative to the free-field ground surface motion. The results of a probabilistic seismic hazard analysis, whether in the form of a response spectrum or acceleration time series for dynamic analysis, do not include the influence of these kinematic effects, and must be modified for input to structural analysis. One approach is to use the "effective support motion" corresponding to the free-field ground motion at a certain depth below the ground surface, since the motion at depth typically has lower amplitude than the surface motion. This procedure is described in Kavazanjian et al. (2011). Alternatively, Turner et al. (2017) provide simple models for modifying free-field response spectra or acceleration time series to account for kinematic interaction based on the results of a parametric nonlinear analysis study covering a wide range of soil, foundation, and ground motion conditions.

9.5 SUMMARY

This chapter outlines methods for the analysis and design of individual vertical drilled shafts for lateral loads, including a simple example. The recommended method uses p-y curves to model the nonlinear relationship of soil resistance as a function of lateral displacement along the length of the shaft. This method has a history of use for transportation and offshore structures, and also provides information

needed for structural design of drilled shafts (covered in detail in Chapter 12). The application of the method with LRFD design concepts is outlined, along with some guidelines for selection and use of appropriate p-y soil models for foundations.

In the overall design process, lateral loading conditions often dictate the diameter of the drilled shaft that will be required and therefore a consideration of this aspect of design is typically required early in the planning and preliminary design process. It is possible that lateral load requirements may control drilled shaft length; however, axial load demands are more likely to dictate final tip elevations for drilled shafts, as outlined in Chapter 10.

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CHAPTER 10
GEOTECHNICAL DESIGN FOR AXIAL LOADING

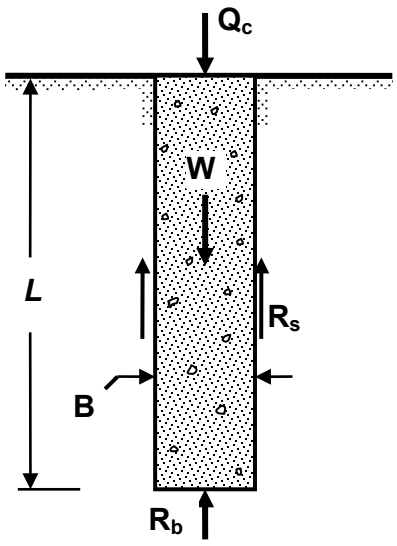
Drilled shafts provide a highly effective means to transmit axial compression and uplift loads to the ground. Design for axial loading requires analysis of strength and service limit states for compression and uplift, and may also require evaluation of extreme event limit states including earthquake. This chapter presents recommendations for design under axial compression and uplift loading, and includes a step-by-step design procedure for this part of the design process.

Design methods and equations presented herein are consistent with those presented in the AASHTO LRFD Bridge Design Specifications (AASHTO, 2017a). For design issues not addressed by AASHTO, or where more recent research provides an improved approach, the differences are noted.

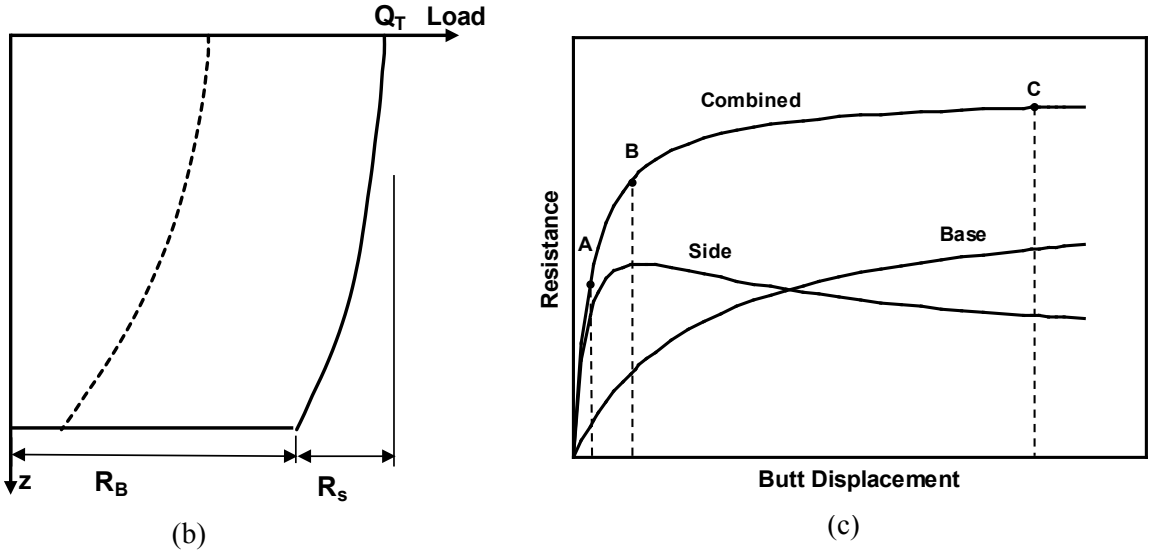
10.1 AXIAL LOAD TRANSFER

The mechanisms of load transfer from a deep foundation to the surrounding ground are fundamental to understanding the basis of design methods for axial loading. The basic load transfer mechanisms were identified through early research on drilled shafts (O'Neill and Reese, 1972) and driven piles (Vesic, 1977). For drilled shafts, the general concepts are summarized by Kulhawy (1991) as illustrated in Figure 10-1, and described as follows. Figure 10-1a illustrates the load transfer behavior of a drilled shaft of length L and diameter B subjected to an axial compression load Q_T applied to the butt (top) of the shaft. Figure 10-1b shows the general relationship between axial resistance and downward displacement. Three components of resistance are shown: (1) side resistance R_S, (2) base (tip) resistance R_B, and (3) combined (total) resistance, Q_T. Figure 10-1c shows the idealized distribution of axial load as a function of depth (z) for different shaft butt displacements. As axial load on the shaft increases from zero, the shaft displaces downward and side resistance in shear is mobilized (Point A in Figure 10-1c). This transfer of load to the surrounding soil or rock results in decreasing load with depth as shown by the dashed curve in Figure 10-1b. At this point, load is transferred predominantly in side resistance and load transmitted to the base may be small. With increasing load, the full side resistance is mobilized (Point B), typically at a displacement of approximately 1/2 inch $\pm$. Further increases in load beyond Point B must be resisted by the base, until the maximum base and combined resistances are reached (Point C). The displacement required to mobilize the maximum base resistance varies, but research suggests that maximum resistance is reached at a displacement equivalent to about 4 to 5 percent of the shaft diameter for bearing in cohesive soil or rock, and about 10 percent of the shaft diameter for bearing in cohesionless soils. Between Points B and C, side resistance may remain constant or change (increase or decrease) depending upon the stress-strain behavior along the interface between the shaft and soil or rock. In some cases, the shaft continues to exhibit increasing resistance with continued downward displacement, thus a well-defined maximum total load is not defined.

Several important behavioral aspects of drilled shafts are illustrated in Figure 10-1. The first is that side and base resistances develop as a function of shaft displacement, and the peak values of each occur at different displacements. Maximum side resistance occurs at relatively small displacement and is generally independent of shaft diameter. Maximum base resistance occurs at relatively large displacement and is a function of shaft diameter and geomaterial type. Design for service limit states must therefore account for differences in side and base resistance mobilization as a function of axial displacement.



(a)



(b) (c)

Figure 10-1 Generalized Load Transfer Behavior of Drilled Shaft in Compression

10.2 RELATIONSHIP TO OVERALL DESIGN PROCESS

Chapter 8 describes the overall design and construction process, as summarized in the form of the flowchart in Figure 8-1. The information presented in this chapter constitutes a more detailed treatment of Block 11 of Figure 8-1, "Establish Minimum Depth and Diameter for Axial Loads."

Several other steps in the overall process are interrelated with design considerations for axial loading. Block 3 of Figure 8-1, titled "Determine Substructure Loads and Load Combinations at Foundation Level," is carried out in consultation with the project structural engineer and in accordance with AASHTO specifications for design of highway bridges, or other applicable design specifications. AASHTO (2017a)

identifies axial compression and axial uplift of single drilled shafts as strength limit states to be satisfied, and settlement control as a service limit state to be satisfied.

Factored axial force effects (foundation load demands) for each limit state are communicated by the project structural engineer to the foundation designer, and based on structural modeling of the bridge or other structure using the applicable load combinations. Since factored axial load demands transmitted to the foundations will vary for each limit state and for various scour conditions (where applicable), it is important for the foundation designer to understand which limit states and associated axial force demands are applicable to the structure being designed. Not all of the limit states apply in every case.

The service limit state for axial loading is based on settlement criteria for the bridge or other structure. All applicable loads in the Service I Load Combination as specified by AASHTO (2017a) are investigated. For drilled shafts in cohesive soils, transient loads can be omitted from settlement analysis, based on the assumption that the response of cohesive soils to transient loads will not result in significant settlement.

10.3 STEP-BY-STEP PROCEDURE: DESIGN FOR AXIAL LOAD

In this section, a step-by-step design procedure is presented in LRFD format for design of drilled shafts under axial loading. The process is depicted in the simplified flow-chart shown in Figure 10-2. As noted above, this procedure is an expanded description of Block 11 of the overall design process of Figure 8-1. The design steps are summarized as follows:

1. At each foundation location, divide the subsurface strata into a finite number of geomaterial layers; assign one of the following geomaterial types to each layer:
   - Cohesionless soil
   - Cohesive soil
   - Rock

2. Review the strength and service limit states to be satisfied and establish the corresponding axial load combinations and load factors for each foundation.

3. For each geomaterial layer and for each limit state/load case, assign the appropriate geomaterial properties needed for evaluation of axial resistances.

4. Select trial lengths and diameters for initial analyses. The minimum trial shaft diameter may be governed by lateral load considerations (Chapter 9) or structural requirements (Chapter 12).

5. Compute values of nominal unit side resistance for all geomaterial layers through which the trial shaft extends and the nominal unit base resistance at the trial tip elevation.

6. Iterating from Step 4, as necessary, adjust the trial design to satisfy the LRFD requirement for each strength limit state:

$$\sum \eta_i \gamma_i F_i \leq \sum \varphi_i R_i \qquad 10\text{-}1$$

   where the parameters in Equation 10-1 have been defined in Chapter 8. Resistance factors (φ) are selected in accordance with AASHTO guidelines and agency experience.

7. Conduct load-deformation analysis for each trial design and iterate from Step 4, as necessary, to satisfy the LRFD requirement for each service limit state. Service limit state evaluation for axial loading requires analysis of side and base resistances that are mobilized at axial displacement corresponding to the tolerable deformation established for the structure being designed.

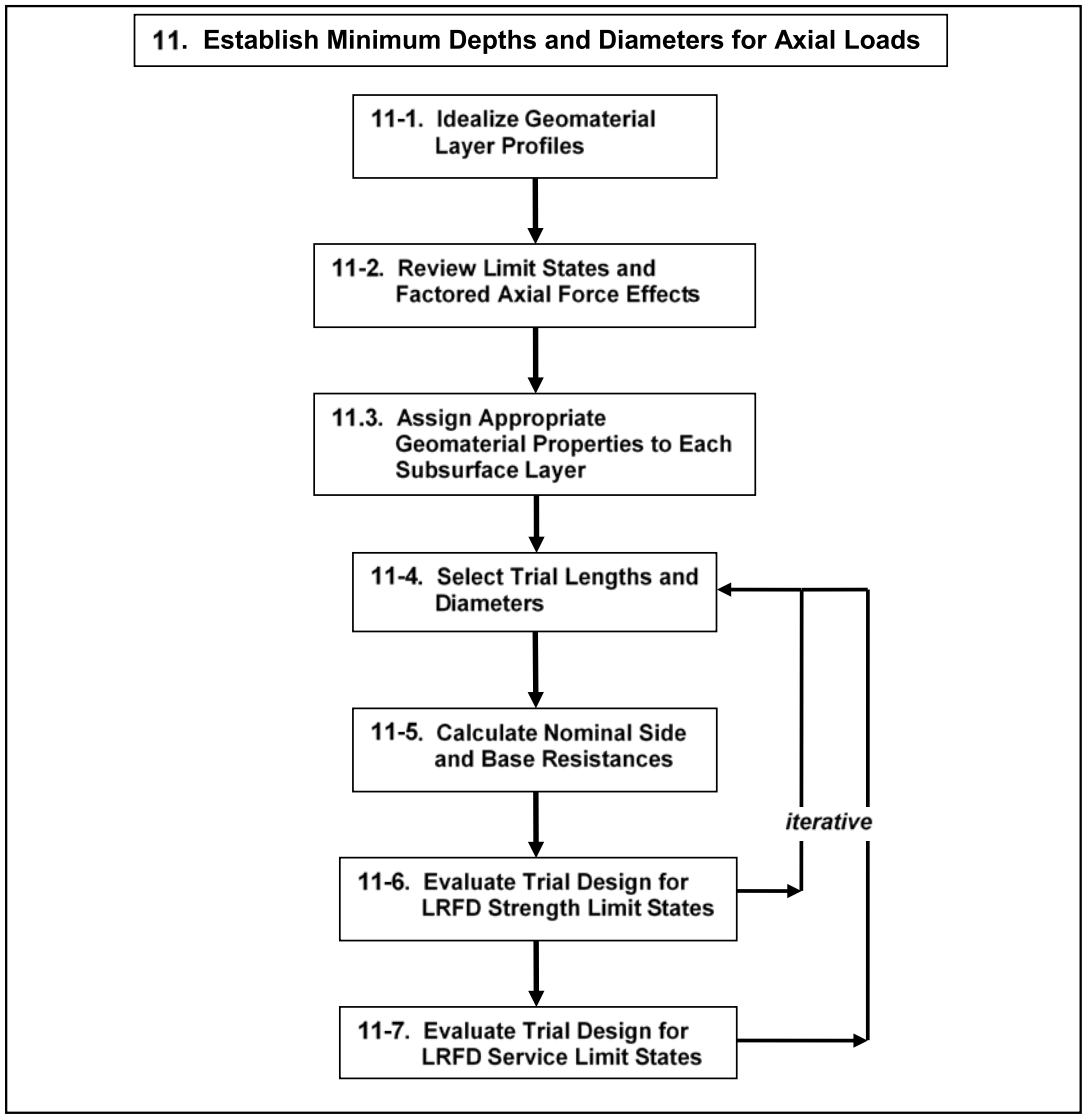


Figure 10-2 Flow Chart of Recommended Steps for Axial Load Design

Results of the above procedure are combined with results of design for other applicable loading modes (lateral, extreme events, structural), and incorporated into the overall design procedure presented in Chapter 8. Details and illustrative examples of each step are presented in the following sections.

10.3.1 Idealized Geomaterial Layer Profiles (Step 1)

A design zone is defined as an area at which one or more drilled shaft foundations will be installed and for which an idealized geomaterial layer profile will be developed. In the case of a bridge, a design zone might

be defined by the location of a single pier to be supported on one or more drilled shafts. If a boring is made at each drilled shaft location, then each shaft may be assigned its own design zone. For each design zone an idealized profile is developed, as illustrated in Figure 10-3. Each layer within the zone is assigned a layer number i, thickness (Δz_i), and geomaterial type. Criteria for assigning material types are as follows:

(a) Cohesionless soil: materials classified as GW, GP, GM, SW, SP, SM, and ML. This includes all gravels and sands with less than 5 percent fines; gravels and sands with silty fines; and non-plastic silts.

(b) Cohesive soil: materials classified as GC, SC, CL, CH, and MH. These include clayey sands and gravels; lean and fat clay soils; and silts with liquid limit over 50.

(c) Rock: cohesive, cemented geomaterial identified as rock on the basis of geologic origin.

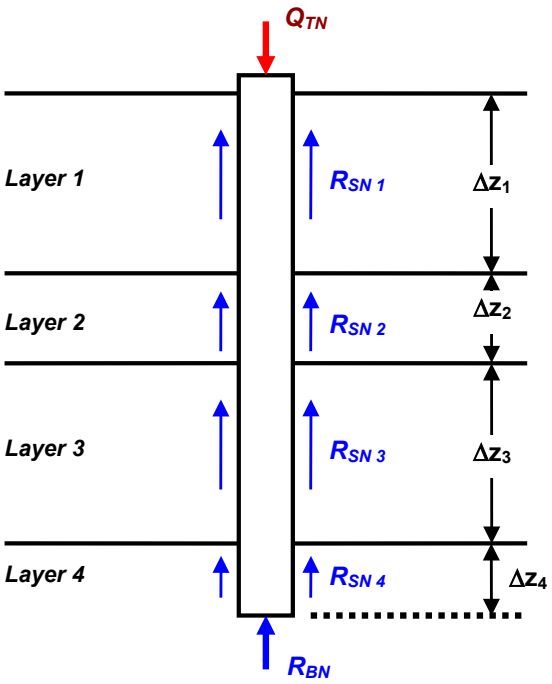


Figure 10-3 Idealized Geomaterial Profile for Computation of Compression Resistances

Some USCS classification groups are difficult to categorize as being cohesionless or cohesive for design purposes. Examples include dual classified groups such as SM-SC, ML-CL, and others. For these materials it may be necessary to measure the strength properties in laboratory direct shear or triaxial tests to determine whether the drained or undrained loading condition will govern its strength and, by inference, whether resistances of drilled shafts in those soil layers should be analyzed for drained or undrained loading. In addition, the wide range of geomaterials and geologic environments encountered in practice means there will always be materials that do not fit the models developed for routine foundation design. Strategies for dealing with some of these special or regionally important geomaterials are presented in Appendix B. Load testing and development of locally-calibrated resistance factors are recommended approaches for dealing with unusual or 'non-textbook' geomaterials.

Scour is to be taken into account when developing idealized subsurface profiles. All drilled shaft resistances considered for strength and service limit states are to be evaluated under scour conditions corresponding to the *scour design flood*. Scour related to a more severe *scour design check flood* is treated as an extreme event. Effects of scour on drilled shaft design include: (1) changes in subsurface stress, (2) reduced embedment and therefore changes in axial and lateral resistances, and (3) possible changes in the

superstructure response and the resulting foundation force effects (loading). Section 10.5 presents an overview of the basic definitions and concepts for evaluating scour. These concepts are applicable to the design of drilled shafts under both lateral and axial loading, and for evaluating scour for the *scour design flood* condition and under the *scour check flood*. Section 10.5.5 describes methods for evaluating effects of scour on axial resistances.

At this stage in the design process, it is recommended that a spreadsheet or other software tool be established in which each geomaterial layer is assigned a number, layer thicknesses, and geomaterial type. In subsequent steps of the design, engineering properties can be assigned to each layer. Side and base resistances can be calculated or entered, and total resistances can be determined for trial designs.

10.3.2 Limit States and Factored Axial Loads (Step 2)

For axial loading, this step involves reviewing the limit states, load combinations and load factors, and scour conditions that pertain specifically to axial loading, as discussed in Section 10.2 of this chapter and in Chapter 8. This step requires direct communication between the structural engineer and foundation designer.

10.3.3 Geomaterial Properties (Step 3)

For each geomaterial layer, the designer assigns the material properties needed to evaluate side and base resistances. TABLE 10-1 summarizes the geomaterial properties needed to evaluate drained and undrained resistances for the primary design methods presented in Section 10.3.5 (Step 5).

TABLE 10-1 GEOMATERIAL PROPERTIES REQUIRED FOR AXIAL RESISTANCES

Geomaterial	Short-term resistance	Long-term (fully drained) resistance
Cohesionless soils	SPT N-values, N_{60} and $(N_1)_{60}$ Average vertical effective stress (σ'_v) for each layer Effective stress friction angle, ϕ'	
Cohesive soils	Undrained shear strength, s_u	[(1)]Effective stress cohesion and friction angle, c' and ϕ'
Rock	Uniaxial compressive strength, q_u RQD Geological Strength Index, GSI	

(1) Axial resistance of drilled shafts in cohesive soils for long-term fully-drained loading is not considered in AASHTO (2017) LRFD Specifications under the assumption that design for undrained loading is adequate for routine practice.

Side and base resistances of drilled shafts in cohesionless soils are evaluated under the assumption of fully drained response, with no need to distinguish between short-term and long-term conditions. Resistances provided by cohesive soil layers are evaluated under the assumption of undrained response to short-term loading (end of construction) and fully drained response to long-term loading. Under most conditions encountered in practice, resistances provided by cohesive soils are evaluated under the assumption that the short-term, undrained resistance is critical, *i.e.*, less than the long-term, fully drained resistance. In these cases, resistances are evaluated in terms of the soil undrained shear strength, s_u. However, there are cases where a designer may wish to consider drained conditions. If a substantial portion of the shaft penetrates

very heavily overconsolidated clay (OCR > 8), where negative porewater pressures can develop in response to short-term loading, there could be good reason to consider designs for both drained and undrained conditions. The analysis resulting in the smaller value of resistance will govern the drilled shaft design.

It is common for a single drilled shaft to derive its resistance to axial loads from several different types of geomaterials, for example when the subsurface profile consists of multiple layers of both cohesive and cohesionless soils. The short-term resistance of the foundation is evaluated in terms of effective stress for cohesionless soil layers (same as long-term), but in terms of total stress for cohesive soil layers. As noted in the previous paragraph, it is assumed in most cases that the short-term, undrained resistance is critical for cohesive soil layers, and analysis is limited to this case. If the long-term, fully-drained response of the shaft is deemed critical, resistances of all geomaterial layers are evaluated in terms of effective stress.

Application of LRFD methodology is based on the use of mean values of engineering properties for each geomaterial layer providing axial resistance. When the parameters used to evaluate resistance (*e.g.*, N_{60}, s_u, q_u) are presented in a spreadsheet or other software format, computation of mean values, standard deviation, and coefficient of variation (COV%) is convenient and efficient. It is also useful to construct graphs showing the variation of each geomaterial property as a function of depth and to plot a linear trend line. The value at layer mid-depth (mean value) is the value selected for design. The COV must also be less than a specified value in order to apply the resistance factors presented in Table 8-4. A recommended upper limit on the COV of N-values within a single geomaterial layer is 45 percent. An upper limit on undrained shear strength of a cohesive soil layer (based on CU or UU triaxial tests) is 35 percent. Otherwise, additional data are required to reduce the variability associated with the geomaterial properties (reduce the COV), or the resistance factors must be reduced based on engineering judgment. For layers in which the variation in parameters exceed the above limits, consideration should be given to splitting the layer into two or more sub-layers.

10.3.4 Select Trial Lengths and Diameters (Step 4)

This step involves selection of trial depths and diameters of drilled shafts to be evaluated for axial force effects associated with each limit state established in Step 2. If trial designs have been established on the basis of lateral loading (Chapter 9) these trial dimensions provide a starting point. Experience in drilled shaft design and thorough knowledge of the ground conditions are needed to make reasonable approximations for initial trial designs. Several rules of thumb and general principles for selecting initial shaft geometries include the following:

- Length to diameter ratios (L/B) are generally in the range of 3 to 30
- In soil, depths to 100 ft and diameters up to 8 ft are considered routine (for construction) in most cases
- Depths to 200 ft and diameters up to 12 ft are within the range of well-equipped specialty subcontractors, but present increased risks during construction
- Rock sockets up to 50 ft are possible (but seldom warranted); common diameter of sockets $B \leq 5$ ft
- Rock sockets up to 8 ft in diameter and larger are within the range of well-equipped, experienced specialty subcontractors
- Deeper and larger diameter shafts are always possible, but may be at premium costs and may pose major construction challenges
- If depth can be limited to allow the dry method of construction, costs and inspection effort are generally more favorable

- Where boulders or rock fragments are present, a larger diameter may be more favorable for material removal
- Structural considerations may govern shaft dimensions when it is desirable to match the diameter of the shaft and the structural column (typically the shaft diameter is slightly larger than the column to allow for drilled shaft installation tolerance); larger diameter may be needed to reduce congestion of steel reinforcement; reinforcement may be continuous between the shaft and column, or it may be spliced. Close consultation with the project structural engineer is required to address these issues
- Shaft diameter should provide adequate concrete cover for the steel reinforcing bars. Minimum cover ranges from 3 to 6 inches, depending on shaft diameter, as described in Chapter 6 and in accordance with project specifications (Chapter 14).
- Shaft diameters in soil overburden should be at least 6 inches larger than the required diameter of the rock socket to facilitate operation of rock drilling equipment
- For sign structures, the shaft diameter should be large enough to accommodate the bearing or anchor plate.

Sites underlain by rock often present a variety of design choices that can significantly affect cost and constructability. If rock exists within the practical depth of excavation, a decision must be made whether to place the base of the shaft on top of the rock surface, into the rock (a socket), or to "float" the drilled shaft above the rock formation. Where adequate resistance can be developed to satisfy all of the relevant limit states, and where scour is not an issue, the most economical design usually is one that avoids excavation of rock by either floating the shaft or locating the tip at the top of rock. An exception would be the case in which the drilled shaft would not develop adequate resistance even with a very large diameter (say, up to 12 ft) and rock is relatively close to the ground surface.

The decision of whether to bear on rock or socket into rock is based on (*i*) quality of the rock near its interface with the overburden material, (*ii*) whether the rock is sloping severely, and (*iii*) whether adequate resistance can be developed without a socket. If the rock is highly weathered, karstic, or sloping severely, or if the overburden can be scoured down close to the rock surface, a socket is usually used. Otherwise, restricting excavation into the rock can result in cost savings relative to using a socket. If the rock is massive and hard (for example, $q_u \geq$ 10,000 psi), socket excavation will proceed slowly and construction costs will be high. In such a case, it may be reasonable to specify a drilled shaft of relatively large diameter and position its base on the surface of the rock, or perhaps 6 to 12 inches into the rock to allow for making a seal with a casing, rather than designing for a smaller-diameter socket.

A designer must often decide whether to utilize a single large-diameter shaft or divide the load among two or more shafts in a group. In general, the single shaft option is more cost effective, but there are exceptions. For example, if access is difficult, such as on a steep slope, equipment for installing smaller diameter shafts in a group may be more feasible and cost-effective than use of large equipment that may require construction of temporary retaining walls or other slope stabilization measures, or a trestle. In rock, multiple, small-diameter sockets may be more constructible and cost-effective than a single large-diameter socket. In some cases the only way to establish the most cost-effective design is to consider both a single-shaft option and trial designs involving multiple shafts. These options can then be compared on the basis of cost and constructability. Further issues associated with groups of drilled shafts are discussed in Chapter 11.

10.3.5 Calculate Nominal Side and Base Resistances (Step 5)

This step involves calculation of the geotechnical resistances for axial compression or uplift loads. The methods presented herein were selected to be consistent with those presented in Article 10.8 of the 2017 AASHTO LRFD Bridge Design Specifications, unless research shows that improved methods supported by data from full-scale axial load testing warrants a different approach.

Considering the two components of resistance for axial compression loading (side and base), the summation of factored resistances (right side of Equation 10-1) for evaluation of LRFD strength limit states is given by:

$$\sum \varphi_i R_i = \sum_{i=1}^{n} \varphi_{S,i} R_{SN,i} + \varphi_B R_{BN} \qquad 10\text{-}2$$

where:

$R_{SN,i}$ = nominal side resistance for layer *i*,
$\varphi_{S,i}$ = resistance factor for side resistance in layer *i*,
n = number of layers providing side resistance,
R_{BN} = nominal base resistance, and
φ_B = resistance factor for base resistance.

Nominal side resistance for a specific geomaterial layer is the product of the nominal unit side resistance (f_{SN}) and the cylindrical surface area over which side resistance develops, expressed as the product of the layer thickness (Δz_i) and the shaft circumference, or:

$$R_{SN} = \pi B \Delta z_i f_{SN} \qquad 10\text{-}3$$

where:

B = shaft diameter,
Δz_i = thickness of layer i, and
f_{SN} = nominal unit side resistance.

Nominal unit side resistance is evaluated in terms of effective stress for cohesionless soil layers. Nominal unit side resistance in cohesive soil layers is evaluated in terms of total stress for end-of-construction (undrained) conditions. If the long-term (fully-drained) side resistance in cohesive soil is deemed to be important, effective stress analysis should be conducted. Nominal unit side resistance in rock is evaluated in terms of uniaxial compressive strength.

Nominal base resistance is the product of the nominal unit base resistance (q_{BN}) and the cross-sectional area of bearing at the shaft base (A_{base}), or:

$$R_{BN} = \frac{\pi B^2}{4} q_{BN} \qquad 10\text{-}4$$

Methods for evaluating unit side and base resistances are presented below for each category of geomaterial.

10.3.5.1 Cohesionless Soils

Side Resistance

The nominal side resistance of a drilled shaft in cohesionless soil can be expressed as the frictional resistance that develops over a cylindrical shear surface defined by the soil-shaft interface. As illustrated in Figure 10-4., the unit side resistance is directly proportional to the normal stress acting on the interface. By Equation 10-3, nominal side resistance is then given by:

$$R_{SN} = \pi B \Delta z \, f_{SN} = \pi B \Delta z \left(\sigma'_v \, K \tan\delta\right) \qquad 10\text{-}5$$

where:

R_{SN} = nominal side resistance
B = shaft diameter
Δz = thickness of the soil layer over which resistance is calculated
σ'_v = average vertical effective stress over the depth interval Δz
K = coefficient of horizontal soil stress ($K = \sigma'_h / \sigma'_v$)
σ'_h = horizontal effective stress
δ = effective stress angle of friction for the soil-shaft interface

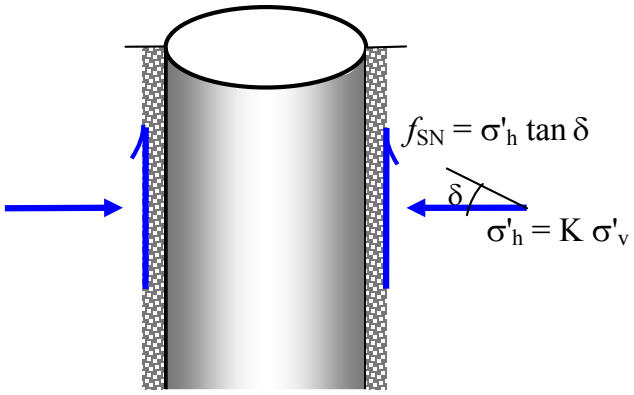


Figure 10-4 Frictional Model of Unit Side Resistance, Drilled Shaft in Cohesionless Soil

For convenience, the following terms may be combined:

$$\beta = K \tan\delta \qquad 10\text{-}6$$

and

$$f_{SN} = \sigma'_v \, \beta \qquad 10\text{-}7$$

in which β = side resistance coefficient (hence the term "beta method") and f_{SN} = nominal unit side resistance. A rational approach developed by Chen and Kulhawy (2002) is to evaluate separately the values of K and δ which are then combined to determine β. Research published over the past 25 years demonstrate

that this approach can provide reliable estimates of side resistance and represents a rational method to incorporate soil strength and state of stress into design equations for side resistance in cohesionless soils.

The operative value of K, coefficient of horizontal soil stress, is a function of the in-situ (at-rest) value, K_o, and changes in horizontal stress that occur in response to drilled shaft construction, given by the ratio K/K_o. A rational first-order approximation is that $K/K_o = 1$, assuming there is no stress change induced by construction. For simple virgin loading-unloading of "normal soils" that are not cemented, the K_o value increases with overconsolidation ratio (OCR) and can be approximated according to (Mayne and Kulhawy, 1982):

$$K_o = (1 - \sin \phi') \, OCR^{\sin \phi'} \leq K_p \qquad 10\text{-}8$$

$$OCR = \frac{\sigma'_p}{\sigma'_v} \qquad 10\text{-}9$$

where σ'_p = effective vertical preconsolidation stress. Note that the value of K_o as given by Equation 10-8 is limited to an upper-bound value corresponding to the coefficient of passive earth pressure, which, for a cohesionless soil, is given by:

$$K_p = \tan^2\left(45^\circ + \frac{\phi'}{2}\right) \qquad 10\text{-}10$$

A variety of methods have been proposed for evaluation of either K_o or σ'_p by correlations with in-situ test results. For a practical estimate based on the most commonly used in-situ test (SPT) the following correlation is suggested by Mayne (2007):

$$\frac{\sigma'_p}{p_a} \approx 0.47\left(N_{60}\right)^m \qquad 10\text{-}11$$

where m = 0.6 for clean quartzitic sands and m = 0.8 for silty sands to sandy silts (*e.g.*, Piedmont residual soils), and p_a = atmospheric pressure in the same units as σ'_p (for example, 2,116 psf). Kulhawy and Chen (2007) suggest the following correlation provides a good fit for gravelly soils:

$$\frac{\sigma'_p}{p_a} = 0.15\, N_{60} \qquad 10\text{-}12$$

Substituting Equations 10-9 through 10-12 into Equation 10-6 leads to the following approximation of β for cohesionless soils:

$$\beta \approx \left(1 - \sin\phi'\right) \left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'} \tan\phi' \qquad \leq K_p \tan\phi' \qquad 10\text{-}13$$

where σ'_p is estimated by Equation 10-11 for sandy soils and Equation 10-12 for gravelly soils. The value of β at shallow depths should be limited to the value corresponding to a depth of 7.5 ft, which corresponds to a vertical effective stress of approximately 900 psf. At lower confining stress, the correlations for

effective stress friction angle and preconsolidation stress have not been validated and it would be prudent to limit β to the values corresponding to this depth. The value of β evaluated by Equation 10-13 is substituted into Equation 10-7 for determination of unit side resistance, and this value is substituted into Equation 10-5 for determination of nominal side resistance R_{SN} for each layer of cohesionless soil. This model accounts for site-specific variations in horizontal stress and soil strength in a rational manner. The approach is also adaptable to other in-situ methods that allow measurement of horizontal soil stress and its variation with depth, such as pressuremeter test (PMT) and flat plate dilatometer test (DMT). The principal limitation of this approach relates to its reliance on N-values and the correlations employed between N-values, friction angle, and preconsolidation stress. Furthermore, resistance factors have not been established for this method through a probability-based calibration study with AASHTO LRFD load factors. Calibration to allowable stress design (ASD) using a factor of safety of FS = 2.5 yields a resistance factor for side resistance in cohesionless soils of $\varphi_S = 0.55$. Until the proper reliability-based calibration study is conducted, this value is recommended. Agencies are also encouraged to establish resistance factors based on calibration using load tests representative of local or regional conditions.

In the approach described above, it is assumed that no change in horizontal stress, and therefore no change in K, occurs as a result of construction. Experience demonstrates this assumption is valid for dry, slurry (wet-hole), and casing methods of construction with minimal sidewall disturbance, proper handling of slurry and casing, and prompt placement of concrete (Chen and Kulhawy, 2002). However, when these aspects of construction quality are not controlled properly, the coefficient K can be reduced to 2/3 of its initial in-situ value (K_o), or lower if soil caving is allowed to occur. Judgment and understanding of the influence of construction activities are therefore needed to assess the applicability of the design equations to individual projects. The recommended approach is to take the necessary measures that will assure quality of construction, and thereby justifying the use of the design equations presented above. Validation of the design by drilled shaft load tests should also be considered.

When permanent casing is used and extends through layers of cohesionless soil, the basic concepts presented above are valid, with proper consideration of differences in the interface shear strength. AASHTO (2007a) states that no specific data are available, but that casing reduction factors of 0.60 to 0.75 are commonly used. A common practice is to specify permanent casing in very soft soils and in subsurface zones where scour is expected, in which case side resistance may be neglected over this depth.

For each strength or service limit state considered, side resistance in cohesionless soils must account for scour resulting from the design flood, where applicable. The most significant effect is that all material above the total scour line is assumed to be removed and unavailable for axial support. Changes in subsurface stress also occur in response to removal of soil, and these changes will affect side resistance calculated by the β-method. This issue is considered in Section 10.5.

Illustrative Example 10-1 on the following page demonstrates determination of unit side resistance by the β-method as presented above.

Illustrative Example 10-1: Side Resistance in Cohesionless Soil Layers by β-Method

Figure 10-5 shows the idealized subsurface profile (Step 1), soil properties (Step 3), and trial dimensions of a drilled shaft (Step 4) under consideration for a bridge project. Calculate the nominal side resistance for the two layers of silty sand extending to a depth of 33 ft using the β-method as described above.

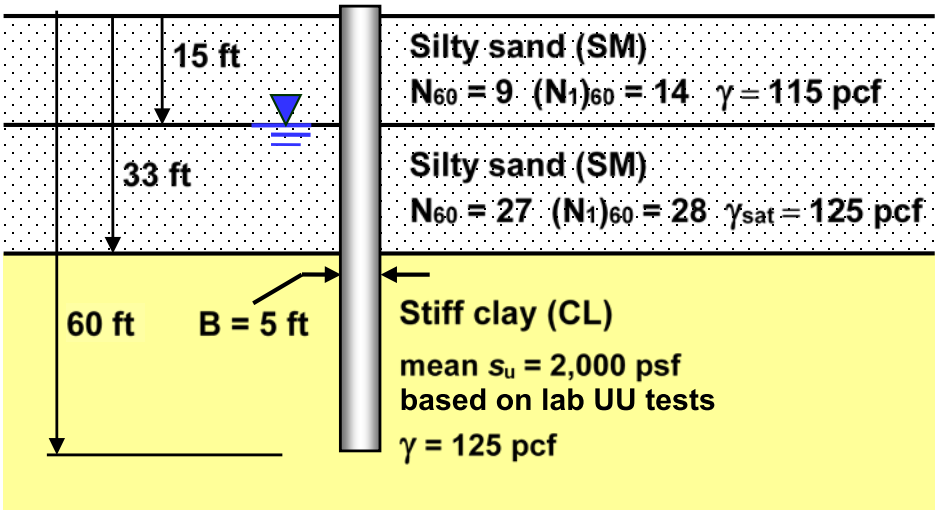


Figure 10-5 Subsurface Profile and Drilled Shaft for Illustrative Example 10-1, 10-2, 10-3, and 10-5

Layer 1 extends from the ground surface to a depth of 15 ft. Layer 2 extends from 15 to 33 ft. The table below identifies each layer along with its thickness, depth to mid-layer, and vertical effective stress at mid-layer depth.

Layer	Depth Interval (ft)	Layer Thickness (ft)	Depth to Mid-point of Layer (ft)	Vertical Effective Stress at Mid-Depth (psf)
1	0 - 15	15	7.5	862.5
2	15- 33	18	24	2,288.4

Effective Vertical Stress at Mid-Depth:

Layer 1: σ'_v = 7.5 ft (115 pcf) = 862.5 psf

Layer 2: σ'_v = 15 ft (115 pcf) + 9 ft (125-62.4 pcf) = 2,288.4 psf

Friction Angle:

Layer 1 $\phi' = \delta = 27.5 + 9.2\log[14] = 38^\circ$ (see Chen and Kulhawy, 2002)

Layer 2 $\phi' = \delta = 27.5 + 9.2\log[28] = 41^\circ$

Preconsolidation Stress:

Layer 1 $\sigma'_p \approx 2{,}116 \times 0.47(9)^{0.8} = 5{,}768\ psf$

Layer 2 $\sigma'_p \approx 2{,}116 \times 0.47(27)^{0.8} = 13{,}890\ psf$

Overconsolidation Ratio:

Layer 1 $OCR = \frac{5{,}768}{862.5} = 6.7$

Layer 2 $OCR = \frac{13{,}890}{2{,}288.4} = 6.1$

At-Rest Coefficient of Lateral Earth Pressure:

Layer 1 $K_o = (1 - \sin 38^o)\, 6.7^{\sin 38} = 1.24 \leq K_p\ (= 4.20)$

Layer 2 $K_o = (1 - \sin 41^o)\, 6.1^{\sin 41} = 1.13 \leq K_p\ (= 4.81)$

Beta:

Layer 1 $\beta = K \tan\delta = 1.24 \tan(38^o) = 0.97$

Layer 2 $\beta = K \tan\delta = 1.13 \tan(41^o) = 0.98$

Unit side resistance:

Layer 1 $f_{SN} = \sigma'_v \beta = 862.4$ psf $(0.97) =$ **837 psf**

Layer 2 $f_{SN} = \sigma'_v \beta = 2{,}288.4$ psf $(1.08) =$ **2,248 psf**

Results of the above calculations are summarized by layer in the following table.

Layer	ϕ' (degrees)	σ'_p (psf)	OCR	K_o	β	f_{SN} (psf)
1	38	5,768	6.7	1.24	0.97	837
2	41	13,890	6.1	1.13	0.98	2,248

Finally, using Equation 10-3 the nominal side resistance provided by each layer is computed:

$R_{SN,1} = \pi$ (5 ft) 15 ft x 837 psf=197,213 lb= 197.2 kips

$R_{SN,2} = \pi$ (5 ft) 18 ft x 2,248 psf=698,659 lb= 635.6 kips

The summation of nominal side resistances for the two silty sand layers is therefore:

ΣR_{SN}=197.2 + 635.6=**833 kips**

Base Resistance

Base resistance in cohesionless soils develops as a function of downward displacement (Figure 10-1c). Observations from load tests show that the downward displacement required to mobilize the full base resistance varies widely. This behavior can be attributed to the influence of drilled shaft construction on soil properties and stresses beneath the base. The process of excavation releases some of the stress in the soil beneath the base. The magnitude of stress release increases with increasing depth and may allow heave, lateral flow, and possibly upward groundwater seepage, all of which will decrease soil strength and stiffness from the initial in-situ state. The effectiveness of base cleaning may also affect base load-deformation behavior, particularly if loose excavated soils (cuttings) are present at the base. Bearing capacity theory for cohesionless soils provides an analytical solution for the maximum unit bearing stress that can be developed. However, when changes in soil properties as a result of stress release and base cleaning are considered, theoretical evaluation of bearing capacity becomes less reliable. Therefore, direct empirical correlations between SPT N-values and mobilized base resistance determined from load tests provide a more pragmatic approach. The following correlation developed by Reese and O'Neill (1989) is recommended for routine design:

$$q_{BN}\ (\text{tsf}) = 0.60\, N_{60} \leq 30\ \text{tsf} \qquad 10\text{-}14$$

in which q_{BN} = nominal unit base resistance and N_{60} is the average value between the base and two diameters beneath the base. Equation 10-14 is based on measured base resistances from compression load tests on drilled shafts with clean bases at settlements equal to five percent of the base diameter. The upper limit of 30 tsf corresponds to the largest values observed in the load tests used to develop the correlation. Note that the N-value to be used in Equation 10-14 is the field value, with no correction for overburden stress, although it is recommended that N-values correspond to 60 percent hammer efficiency. Equation 10-14 is also limited to cohesionless soils with SPT N-values of 50 blows per foot or less. In cohesionless soils with N-values exceeding 50, load testing is recommended; otherwise unit base resistance is limited to the upper-bound value indicated in Equation 10-14. Agencies are also encouraged to develop in-house correlations and to conduct calibration studies to establish resistance factors appropriate for local conditions and construction practices.

An important point is noted regarding the use of Equation 10-14 to define a strength limit state. Since the equation predicts the base resistance developed at a defined value of displacement, it could be argued that Equation 10-14 does not define a limit based on strength, but rather on displacement. Nevertheless, the calibration studies conducted by Barker et al. (1991), Paikowsky et al. (2004), and Allen (2005) are based on Equation 10-14. According to Allen (2005), the current resistance factor of $\varphi = 0.50$ specified in AASHTO (2007) was established on the basis of fitting to a factor of safety FS = 2.75 with consideration of the reliability-based factors reported by Paikowsky et al. (2004).

As an alternative to the empirical relationship given by Equation 10-14, bearing capacity theory can be applied to the calculation of nominal base resistance of a drilled shaft bearing in cohesionless soil. For the case of bearing in cohesionless soils, the deformation required for mobilization of the full base resistance can vary over a wide range as noted above. For this reason, values based on the simple expression of Equation 10-14 are recommended for routine design.

10.3.5.2 Cohesive Soils

Side Resistance

Short-term undrained side resistance in cohesive soil layers is evaluated in terms of undrained shear strength. Equation 10-3 then becomes:

$$R_{SN} = \pi B \Delta z f_{SN} = \pi B \Delta z (\alpha s_u)_i \qquad 10\text{-}15$$

where:

R_{SN} = nominal side resistance,
B = shaft diameter,
Δz = thickness of the soil layer over which resistance is calculated,
s_u = average undrained shear strength over the depth interval Δz,
α = coefficient relating unit side resistance to undrained shear strength (hence the term "alpha method"), and
f_{SN} = nominal unit side resistance.

Key terms to evaluate in Equation 10-15 are the mean undrained shear strength of the cohesive soil layer and the coefficient α.

Whenever possible, laboratory strength testing of undisturbed samples is the recommended method for measurement of undrained shear strength, s_u. In-situ testing can be used to assess s_u at sites where collection of undisturbed samples is difficult, but when in-situ tests are used it is strongly recommended that the results be calibrated against laboratory tests.

Laboratory strength tests for measurement of s_u include the unconfined compression (UC) test, unconsolidated undrained (UU) triaxial compression, and consolidated isotropically undrained triaxial compression (CIUC). Although simpler to perform, the UC and UU tests are not as reliable measures of s_u as CIUC triaxial tests. Effects of sample disturbance, high strain rate, and uncertain drainage conditions in UC and UU tests can provide misleading, but usually conservative, values of undrained strength. The CIUC test is superior because the sample can be reconsolidated to the original in-situ state of stress, providing an undrained strength more representative of in-situ stress conditions. Both AASHTO (2017a) and GEC No. 5 (Loehr et al., 2017) note the benefits of CIUC triaxial compression tests over UC or UU tests for determination of s_u, while recognizing that typical transportation project practice entails the use of both UU and UC testing, and for cases where undisturbed sampling is difficult, vane shear testing (VST).

For values of undrained shear strength determined by UC or UU lab tests, the value of s_u should first be transformed to its equivalent CIUC value using the relationships below recommended by Chen and Kulhawy (1993):

$$\frac{s_u(UC)}{s_u(CIUC)} = 0.893 + 0.513\ log\left(\frac{s_u(UC)}{\sigma'_{vo}}\right) \qquad 10\text{-}16$$

$$\frac{s_u(UU)}{s_u(CIUC)} = 0.911 + 0.499\ log\left(\frac{s_u(UU)}{\sigma'_{vo}}\right) \qquad 10\text{-}17$$

where σ'_{vo} = vertical effective stress.

The adhesion coefficient α is estimated empirically on the basis of load test measurements in which the undrained shear strength (s_u) was measured using laboratory strength tests and unit side resistance was determined from the load test. Figure 10-6 (Chen et al., 2011) shows the recommended relationship between α and the normalized CIUC undrained shear strength, $s_u(CIUC)/p_a$, where p_a = atmospheric pressure in consistent units. A regression curve through the data shown in Figure 10-6 gives the adhesion factor as follows:

$$\alpha = 0.30 + \frac{0.17}{\left(\frac{s_u(CIUC)}{p_a}\right)} \qquad 10\text{-}18$$

The above expression for alpha is recommended as a revision to the expressions for alpha currently given in the AASHTO LRFD Bridge Design Specifications (2017a).

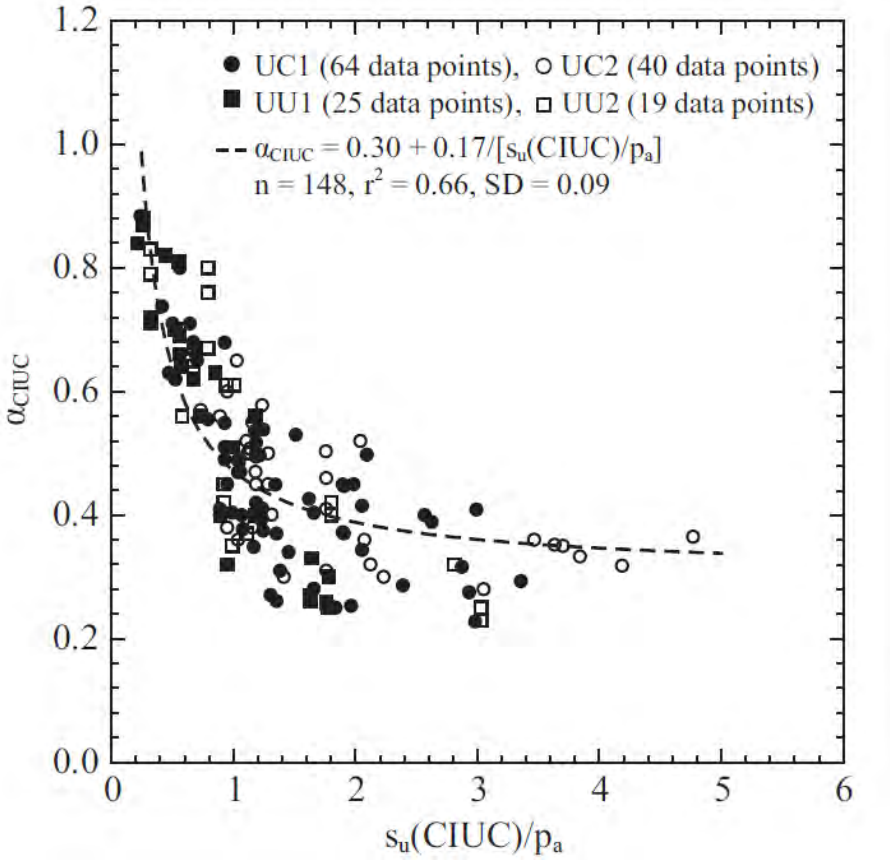


Figure 10-6 Correlation of Side Resistance Coefficient Alpha to Undrained Shear Strength (Chen et al., 2011)

It is common design practice to neglect side resistance over the top 5 ft of a drilled shaft, or to the depth of seasonal moisture change, to account for the potential loss of side resistance as soil expands and contracts in response to wetting and drying, freezing and thawing, "gapping" caused by cyclic lateral loading, or any process occurring near the ground surface having the potential to soften the soil or eliminate contact between the shaft and soil. This recommendation is subject to modification based on local experience and judgment of the designer.

Illustrative Example 10-2 demonstrates the calculation of nominal side resistance for a drilled shaft in cohesive soil by the α-method, using the subsurface profile defined in Figure 10-5.

When permanent casing is used and extends through layers of cohesive soil, side resistance will generally be reduced relative to that of an uncased shaft. Reduction factors applied to steel piles compared to driven concrete piles range from 50 to 75 percent; this range can be used as a guide in selecting a reduction for cased drilled shafts, although no specific data are available to support recommendations for reduction factors. It is anticipated that the reduction factor for drilled shafts may be near the lower end of the range since the comparison is between a rough concrete surface and a smooth steel surface, whereas for driven piles the formed concrete surface is smooth.

Illustrative Example 10-2: Side Resistance in Clay

Consider the soil profile and trial drilled shaft dimensions of Illustrative Example 10-1. The lower 27 ft of the trial shaft penetrates a stiff clay layer with a mean undrained shear strength of 2,000 psf based on UU triaxial tests performed on Shelby tube samples. Using the α-method presented above, calculate the nominal side resistance provided by the clay stratum represented by Layer 3.

Estimate s_u(CIUC) from the UU mean strength:

Equation 10-17 requires the average vertical effective stress in the cohesive soil layer. Referring to the subsurface profile, the depth to mid-layer is 33 + 27/2 = 46.5 ft, and:

σ'_{vo} = 15(115) + 31.5(125-62.4) = 3,697 psf

s_u/σ'_{vo} = 2,000/3,697 = 0.541

$$s_u(CIUC) = \frac{2{,}000\ psf}{0.911+0.499\ log(0.541)} = 2{,}571\ psf$$

Then by Equation 10-18:

$$\alpha = 0.30 + \frac{0.17}{\left(\frac{2{,}571}{2{,}116}\right)} = 0.44$$

Unit side resistance:

f_{SN} = α s_u(CIUC) = 0.44 (2,571 psf) = 1,131 psf

Nominal side resistance:

Layer 3: $R_{SN,3}$ = π (5 ft) (27 ft) 1,131 psf = 479,674 lb = **479.7 kips**

An implicit assumption associated with use of the α-method as described above is that design of drilled shafts in cohesive soils for short-term undrained loading by total stress analysis is adequate for routine design. There is no evidence or record of unsatisfactory performance to suggest otherwise. However, a designer may choose to consider the fully-drained side resistance for drilled shafts in very heavily overconsolidated clay (OCR > 8), based on the concept that the long-term strength of these soils is lower than the short-term undrained strength. In this case, effective stress analysis is conducted in the same manner as described above for cohesionless soils, using Equation 10-5. The parameters used in Equation 10-5 are evaluated differently. It is normally assumed that effective stress cohesion (c') is zero in response to the drilling process. Values of effective stress friction angle (ϕ') should be measured in laboratory triaxial tests, either consolidated-drained (CD) or consolidated-undrained (CU) with porewater pressure measurements. The interface friction angle can be taken equal to ϕ' The value of K can be taken equal to K_o as determined from appropriate in-situ measurements or on the basis of overconsolidation ratio as described in Chapter 2. No recommendation is made herein or in AASHTO (2017a) for the resistance factor to apply to the fully-drained side resistance in cohesive soils.

Base Resistance

Bearing capacity theory applied to the case of a deep foundation bearing on a cohesive soil, in terms of total stress analysis, yields the following approximate expression which is sufficient for design (O'Neill and Reese, 1999):

$$q_{BN} = N^*_c \, s_u \qquad 10\text{-}19$$

where N^*_c = bearing capacity factor and s_u = mean undrained shear strength of the cohesive soil over a depth of 2*B* below the base. For cases where the shaft depth is at least 3 times the diameter and the mean undrained shear strength is at least 2,000 psf, the bearing capacity factor can be taken as 9.0. For smaller values of undrained shear strength, N^*_c can be approximated as a function of undrained shear strength as given in TABLE 10-2. Linear interpolation can be used for values between those tabulated. Note that it is unusual to locate the base of a drilled shaft in cohesive soil with s_u less than 2,000 psf when compression loads are supported.

TABLE 10-2 BEARING CAPACITY FACTOR N^*_C

Undrained shear strength, s_u (lb/ft^2)	$I_r \approx \frac{E_u}{3s_u}$	N^*_c
500	50	6.5
1,000	150	8.0
≥ 2,000	250 - 300	9.0

E_u = Undrained Young's Modulus

For drilled shafts with depth of embedment less than three times the diameter (D < 3*B*) the following expression applies:

$$q_{BN} = \frac{2}{3}\left[1 + \frac{1}{6}\left(\frac{D}{B}\right)\right] N^*_c \, s_u \qquad 10\text{-}20$$

where N^*_c is approximated from Table 10-2.

Illustrative Example 10-3 demonstrates the computation of base resistance using the method described above. The resistance value of φ = 0.40 (Table 8-4) for base resistance by the above method is based on the report by Allen (2005) and was established by a combination of fitting to the ASD factor of safety (FS = 2.75) and taking into account the reliability-based analysis conducted by Paikowsky et al. (2004).

A designer may wish to evaluate the fully-drained base resistance of drilled shafts in cohesive soils using effective stress analysis. As noted above for side resistance, fully-drained response may be considered for shafts bearing on heavily overconsolidated clay soils (OCR > 8). In this case base resistance can be analyzed in terms of bearing capacity theory. The analysis requires knowledge of the effective stress strength properties (*c*' and ϕ') which should be determined by appropriate laboratory tests (CD or CU with pore water pressure measurements, triaxial compression). Effective stress analysis with bearing capacity theory is not addressed in current AASHTO (2017a) specifications.

Illustrative Example 10-3: Base Resistance in Clay

For the soil profile and trial drilled shaft of Illustrative Example 10-1 as shown in Figure 10-5, calculate the nominal base resistance R_{BN}.

From Illustrative Example 10-2, the equivalent CIUC undrained shear strength of the clay is 2,571 psf. Since the trial shaft depth (60 ft) is more than 3 times its diameter (5 ft) and s_u is greater than 2,000 psf, the bearing capacity factor N^*_c can be taken as 9.0 (Table 10-2). By Equation 10-19 the nominal unit base resistance is:

$$q_{BN} = 9.0\ (2{,}571\ \text{psf}) = 23{,}139\ \text{psf} = 23.1\ \text{ksf}$$

and the nominal base resistance is:

$$R_{BN,3} = \pi/4\ (5\ \text{ft})^2\ x\ 23.1\ \text{ksf} = \mathbf{\underline{453.6\ kips}}$$

10.3.5.3 Rock

Side Resistance

Unit side resistance for shafts in rock may be evaluated on the basis of mean uniaxial compressive strength of the rock, as follows:

$$\frac{f_{SN}}{p_a} = C\sqrt{\frac{q_u}{p_a}} \qquad 10\text{-}21$$

in which:

q_u = mean value of uniaxial compressive strength for the rock layer,
p_a = atmospheric pressure in the same units as q_u, and
C = a regression coefficient used to analyze load test results.

Studies relating side resistance to rock compressive strength include those of Horvath and Kenney (1979), Rowe and Armitage (1987), Kulhawy and Phoon (1993), and others. The most recent regression analysis of available load test data is reported by Kulhawy et al. (2005) and yields a mean value of the coefficient C equal to 1.0. Equation 10-21 with $C = 1.0$ is recommended for design of "normal" rock sockets. Kulhawy et al. (2005) emphasize the importance of using values of q_u determined from laboratory uniaxial compression tests in accordance with proper test procedures such as those given by ASTM and on specimens at field moisture contents. Estimating q_u from index tests such as the point load, Schmidt hammer, or others, may be inappropriate due to high levels of variability in correlations with q_u. Use of strength values from samples that have been allowed to dry from their field condition will lead to higher values of q_u and overestimation of foundation resistance.

For design, the value of q_u used in Equation 10-21 should not exceed the compressive strength of the drilled shaft concrete. If, however, load testing can be used to demonstrate side resistances greater than would be calculated using the concrete design strength, the load test-validated values should be allowed, subject to engineering judgment as to how closely the test conditions are representative of production conditions and installation means and methods.

The term "normal" as used above applies to sockets constructed with conventional equipment and resulting in nominally clean sidewalls without resorting to special procedures or artificial roughening. Rock that may be prone to smearing or rapid deterioration upon exposure to atmospheric conditions, water, or slurry, are outside the "normal" range and may require additional measures to insure reliable side resistance. Rock exhibiting this type of behavior, such as clay shales, are discussed further as special geomaterials in Appendix B.

Rock mass that does not allow excavation of an unsupported socket without caving is also outside the "normal" category and may exhibit lower side resistance than given by Equation 10-21 with $C = 1.0$. Rock sockets are nevertheless often installed in rock that caves during excavation, for example by the 'plug-ahead' or 'grout-ahead' method where the caving zone is drilled with a slightly oversized tool, then backfilled with a lean mix which is subsequently re-drilled to the design diameter. For calculating side resistance in cases where caving or instability of the socket excavation is anticipated, the following expression given by O'Neill and Reese (1999) is recommended:

$$\frac{f_{SN}}{p_a} = 0.65\,\alpha_E \sqrt{\frac{q_u}{p_a}} \qquad 10\text{-}22$$

where the coefficient α_E is estimated from the RQD and condition of the discontinuity surfaces of the rock mass. The relationship between RQD, joint condition, and α_E is given in Table 10-3.

TABLE 10-3 SIDE RESISTANCE REDUCTION FACTOR FOR CAVING ROCK

RQD (%)	Joint Modification Factor, α_E	
	Closed joints	Open or gouge-filled joints
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

In summary, for routine design of rock sockets, Equation 10-21 with $C = 1.0$ is recommended. For highly fractured or weathered rock that cannot be drilled without some type of artificial support, such as casing or by the 'plug ahead' method, or where caving is anticipated, Equation 10-22 with the reduction factors given in Table 10-3 based on RQD are recommended. The resistance factor recommended with use of Equations 10-21 and 10-22 is $\varphi = 0.55$ based on fitting to ASD with a factor of safety FS = 2.5, as presented in Table 8-4.

Artificial roughening of rock sockets through the use of grooving tools or other measures can increase side resistance compared to normal sockets. Regression analysis of the available load test data by Kulhawy and

Prakoso (2007) suggests a mean value of C = 1.9 with use of Equation 10-22 for roughened sockets. It is strongly recommended that load tests or local experience be used to verify values of C greater than 1.0. However, the advantages of achieving higher resistance by employing sidewall roughening in conjunction with load testing can potentially justify the cost of load testing.

Base Resistance

Base resistance in rock is more complex than in soil because of the wide range of possible rock mass conditions. Various failure modes are possible depending upon whether rock mass strength is governed by intact rock, fractured rock mass, or structurally controlled by shearing along dominant discontinuity surfaces. In practice, it is common to have information on the uniaxial compressive strength of intact rock (q_u) and the general condition of rock beneath the base of a shaft. Empirical relationships between nominal unit base resistance (q_{BN}) and rock compressive strength can be expressed in the form:

$$q_{BN} = N^*_{cr}\, q_u \qquad 10\text{-}23$$

where N_{cr}^* is an empirical bearing capacity factor for rock.

Studies relating q_{BN} to q_u are reported by Zhang and Einstein (1998) and Prakoso and Kulhawy (2002). There is overlap in the data used in each study, although the authors used different interpretations of load test results to establish q_{BN}. Prakoso and Kulhawy used a consistent definition of limiting base resistance, and limited the data to tests that exhibited a strength limit state according to the L_1-L_2 method. Results of the Prakoso and Kulhawy study are shown in Figure 10-7 in which the bearing capacity factor (N_{cr}^*) is plotted against shaft diameter. The data base included 14 load tests at 9 sites in several rock types, consisting mainly of fine-grained sedimentary rocks. The mean value of N_{cr}^* is 3.38, with a coefficient of variation COV = 35.4%. A lower bound value of N_{cr}^* = 2.5 incorporates most of the points shown in Figure 10-7 and is consistent with work by Rowe and Armitage (1987) in which a value of N_{cr}* = 2.5 is recommended for competent rock. When the data used by Zhang and Einstein are evaluated in the format of Equation 10-23, they yield a mean value of N_{cr}^* = 3.56 and a COV = 61.0%. Considering these three studies, a value of N_{cr}^* = 2.5 is recommended for design when q_u is the sole parameter used for establishing q_{BN} and the following conditions are met:

1. The drilled shaft base is bearing on rock which is either massive or tightly jointed (no compressible seams or joints) to a depth of at least one diameter beneath the base,
2. It can be verified that no solution cavities or voids exist beneath the base, and
3. A clean base can be achieved and verified using conventional clean-out equipment.

Equation 10-23 with the recommended value of N_{cr}^* = 2.5 for bearing on competent rock is based on the original work by Rowe and Armitage (1987). The research cited above validates the use of this equation for routine design in competent rock. The LRFD resistance factor specified in AASHTO (2017a) for use of Equation 10-23 with N_{cr}^* = 2.5 is φ = 0.50, based on fitting to an ASD factor of safety FS = 2.5. Values of N_{cr}^* greater than 2.5, which clearly are possible based on Figure 10-7, are justified when they can be verified by local experience or load testing.

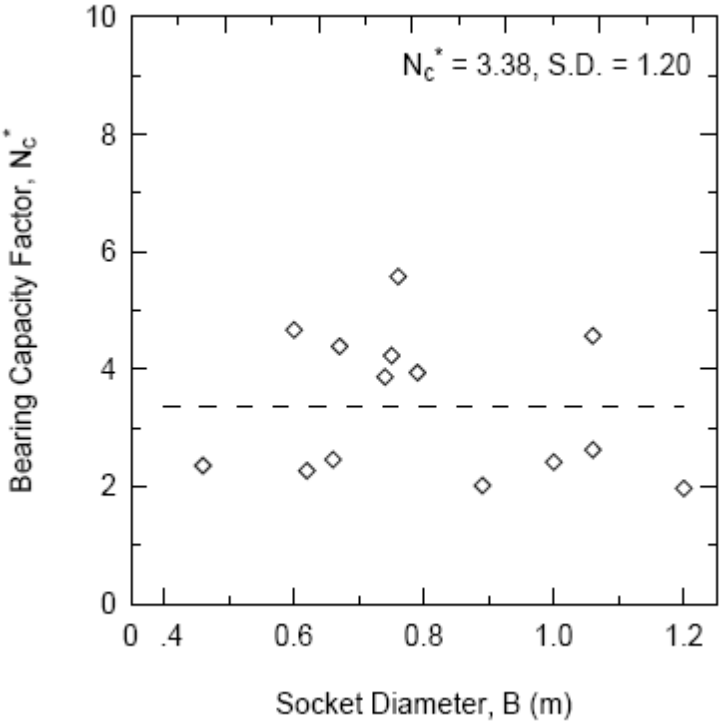


Figure 10-7 Base Resistance Factor for Rock (Prakoso and Kulhawy 2002)

For fractured rock mass, bearing capacity theory in combination with the Hoek-Brown failure criterion provides a framework for evaluating drilled shaft base resistance (Turner and Ramey, 2010). Hoek and Brown (1980) proposed their widely used empirical criterion for characterizing rock mass strength. Based on Hoek et al. (2002) the nonlinear rock mass strength is given by:

$$\sigma_1' = \sigma_3' + q_u\left(m_b\frac{\sigma_3'}{q_u} + s\right)^a \qquad 10\text{-}24$$

where σ'_1 and σ'_3 = major and minor principal effective stresses, respectively, q_u = uniaxial compressive strength of intact rock, m_b is a reduced value of a material constant (m_i) that depends on rock type, and s and a are dimensionless constants that depend on the condition of the rock mass.

The Geologic Strength Index (GSI) provides a means to correlate field and laboratory observations and measurements to the Hoek-Brown strength parameters incorporated into Equation 10-24. The GSI takes into account two fundamental characteristics of a rock mass: the blockiness of the mass and the condition of the discontinuities. In combination with the strength of intact rock (q_u) and the state of stress, these factors control the mechanical behavior of a rock mass. The value of GSI ranges from 0 to 100 with higher values representing more tightly interlocking blocks and tighter/rougher discontinuity surfaces, while lower values indicate more highly fractured rock with poor discontinuity surfaces. Detailed methods for establishing values of GSI from rock core descriptions are beyond the scope of this manual, but can be found in the original references (*e.g.*, Marinos and Hoek, 2000; Hoek et al., 2013) and are also presented in FHWA GEC 5 (Loehr et al., 2017).

The Hoek-Brown strength parameters can be correlated to GSI, as follows. The value of the constant m for intact rock is denoted by m_i and can be estimated based on lithology (see Table 9-10 in GEC 5). Suggested relationships between GSI and the parameters m_b/m_i, s, and a, according to Hoek et al. (2002) are as follows:

$$\frac{m_b}{m_i} = \exp\left(\frac{GSI - 100}{28}\right) \qquad 10\text{-}25$$

$$s = \exp\left(\frac{GSI - 100}{9}\right) \qquad 10\text{-}26$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right) \qquad 10\text{-}27$$

.

Turner and Ramey (2010) show that the Hoek-Brown strength parameters can be incorporated into a bearing capacity analysis, thus providing a means to calculate the nominal base resistance of rock socketed drilled shafts, as follows. The parameter A takes into account the effect of overburden stress and is defined as:

$$A = \sigma'_{vb} + q_u\left[m_b\frac{(\sigma'_{vb})}{q_u} + s\right]^a \qquad 10\text{-}28$$

where σ'_{vb} = vertical effective stress at the socket bearing elevation (tip elevation). The nominal base resistance is then given by:

$$q_{BN} = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a \qquad 10\text{-}29$$

in which the coefficients s, a, and m_b are the Hoek-Brown strength parameters for the intact or fractured rock mass determined as functions of GSI (Equations 10-25 to 10-27).

The value of nominal unit base resistance by Equation 10-29 can be normalized by the rock uniaxial compressive strength to provide a bearing capacity factor N^*_{cr} as defined by Equation 10-23. A value of N^*_{cr} equal to 2.5 can be used as an upper-bound limit to base resistance calculated by Equation 10-23, unless local experience or load tests can be used to validate higher values.

For a given rock type (which determines the value of the parameter m_b) a graph of the bearing capacity factor N^*_{cr} can be prepared showing the relationship between N^*_{cr} and the GSI. To illustrate, Figure 10-8 shows the relationship between N^*_{cr} and GSI for several values of m_i (*i.e*, for several rock types) and for the special case of zero overburden stress. For application to a specific case, similar curves can be prepared taking into account the actual rock type (which determines m_i) and the overburden stress at socket tip elevation.

The recommend resistance factor for application to Equation 10-29 is $\varphi = 0.50$, based on fitting to an ASD factor of safety FS = 2.5.

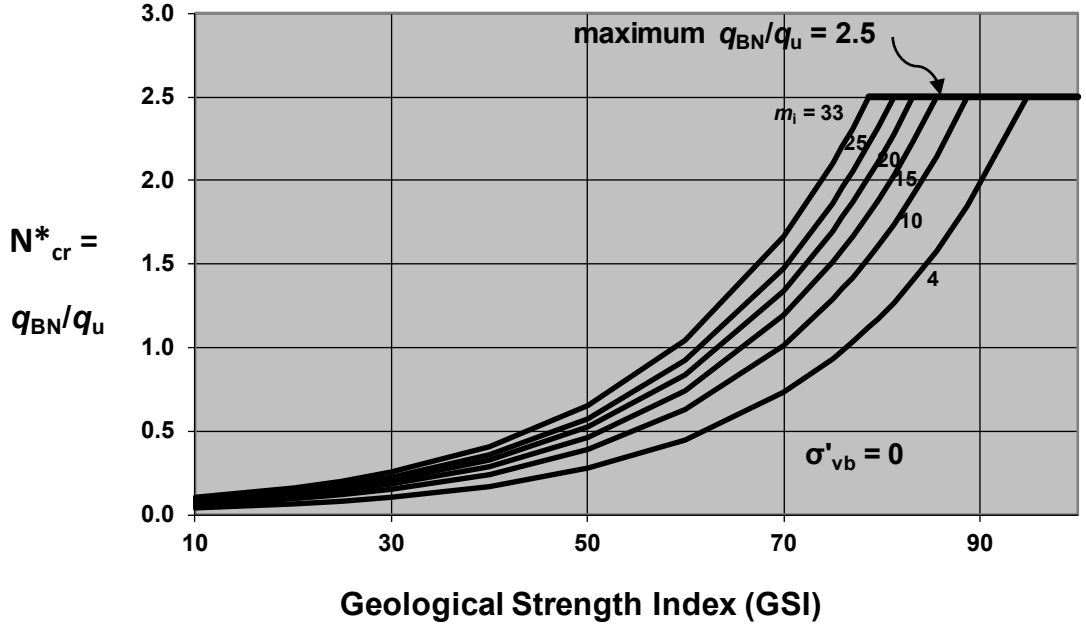


Figure 10-8 Bearing Capacity Factor N^*_{cr} versus GSI for Base Resistance of Fractured Rock Mass

Note that the resistance at the base of a drilled shaft in rock could be governed by either (a) the geotechnical strength of the rock mass beneath the tip, as described above, or (b) the structural strength of the concrete in bearing. Equations for the strength of a concrete member in bearing are presented in Chapter 12. The nominal bearing strength of concrete is a function of its design compressive strength, f_c'. For competent rock with compressive strength q_u approaching or exceeding that of the concrete, the nominal geotechnical base resistance of $2.5q_u$ is likely to be greater than the nominal bearing strength of the concrete shaft. However, the resistance factor applied to the structural strength for bearing ($\varphi = 0.70$) is greater than the geotechnical resistance factor ($\varphi = 0.50$). A comparison of the factored resistances must be carried out on a case-by-case basis to determine whether the geotechnical or structural resistance governs the design base resistance.

Additional Design Considerations for Rock Sockets:

A design decision to be addressed when using rock sockets is whether to neglect one or the other component of resistance (side or base) for the purpose of evaluating strength limit states. With regard to base resistance, AASHTO Article C.10.8.3.5.4a states "Design based on side-wall shear alone should be considered for cases in which the base of 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" (AASHTO, 2017a). The philosophy embraced in the above comment gives a designer the option of neglecting base resistance. However, before making this decision, careful consideration should be given to applying available methods of quality construction and inspection that can provide confidence in base resistance. Construction and quality assurance tools and practices that provide highly effective clean-out at the base of rock sockets, including those constructed by wet methods, are available. For example, air-lifting or pumping from the base in conjunction with inspection tools, such as the Shaft Inspection Device (SID), weighted tape soundings, probing tools, borehole calipers, and others, can be applied more effectively to ensure quality of rock sockets prior to concrete placement (Crapps and Schmertmann, 2002; Turner, 2006). Under most conditions, the cost of quality control and assurance is offset by the economies achieved in socket design by including base resistance. Several State DOT's have utilized load testing to develop confidence in the

use of base resistance in rock formations where base resistance had previously been neglected due to uncertainty.

Reasons cited for neglecting side resistance of rock sockets include (1) the possibility of strain-softening behavior of the sidewall interface, (2) the possibility of degradation of material at the borehole wall in argillaceous rock, and (3) uncertainty regarding the roughness of the sidewall. Brittle behavior along the sidewall, in which side resistance exhibits a significant decrease beyond its peak value, is not commonly observed in load tests on rock sockets. If there is reason to believe strain softening will occur, laboratory direct shear tests of the rock-concrete interface can be used to evaluate the load-deformation behavior and account for it in design. These cases would also be strong candidates for conducting field load tests. Investigating the sidewall shear behavior through laboratory or field testing is generally more cost-effective than neglecting side resistance in the design. Application of quality control and quality assurance through inspection is also necessary to confirm that sidewall conditions in production shafts are of the same quality as laboratory or field test conditions.

Materials that are prone to degradation at the exposed surface of the borehole and are prone to a "smooth" sidewall generally are sedimentary rocks such as shale, claystone, and siltstone. Degradation occurs due to expansion, and opening of cracks and fissures combined with groundwater seepage, and by exposure to air and/or water used for drilling. Hassan and O'Neill (1997) note that in the most severe cases, degradation results in a smear zone at the interface. Smearing may reduce load transfer significantly. As reported by Abu-Hejleh et al. (2003), both smearing and smooth sidewall conditions can be prevented in cohesive IGMs by using roughening tools during the final pass with the rock auger or by grooving tools. Careful inspection prior to concrete placement is required to confirm roughness of the sidewalls. Only when these measures cannot be confirmed would there be cause for neglecting side resistance in design. When new tools are introduced for drilling in rock, inspection of the sidewall for roughness, for example using a feeler bar, is necessary to confirm that the method results in a rough interface.

Illustrative Example 10-4 demonstrates the calculation of side and base resistances in a rock-socketed drilled shaft. The example illustrates the large capacity contributed by base resistance in a relatively small rock socket. To take advantage of the available base resistance, it would be important to specify proper base cleanout methods and to verify base conditions through appropriate inspection.

Illustrative Example 10-4. Rock Socket Resistance for Trial Drilled Shaft Design

A drilled shaft for a major bridge is to be installed through soil and socketed into competent sandstone. A trial design under consideration calls for permanent casing of 6.5 ft diameter extending through the overburden soil and decomposed rock to the top of competent rock. The trial socket is 6 ft in diameter and 10 ft deep, as shown in Figure 10-8. Coring of the sandstone shows RQD values ranging from 37 to 100 in the zone influencing the socket construction and design. Uniaxial compression tests on intact core samples at natural water content yields an average uniaxial compressive strength of q_u = 2,500 psi (360 ksf). The concrete design strength for the trial shaft is f'_c = 4,000 psi (576 ksf). Construction experience in this formation indicates the socket can be excavated with conventional tools and will remain stable without the use of casing.

Compute the nominal side and base resistances for the socketed portion of the trial shaft.

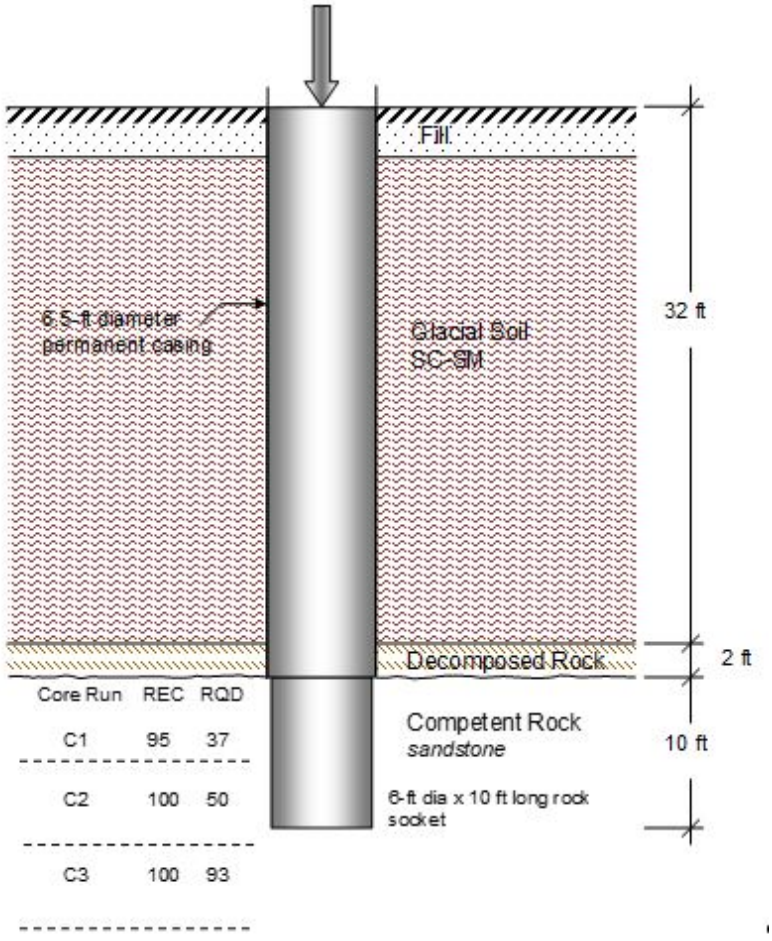


Figure 10-9 Ground Profile and Trial Shaft for Illustrative Example 10-4

The socket excavation satisfies the criteria for a 'normal' socket. Unit side resistance is then calculated by Equation 10-21:

$$f_{SN} = p_a \; x \; C\sqrt{\frac{q_u}{p_a}} = 2.116 \; ksf \; x \; 1.0\sqrt{\frac{360 \; ksf}{2.116 \; ksf}} = 27.6 \text{ ksf}$$

Nominal side resistance:

$$R_{SN,\,socket} = 27.6 \text{ ksf } x \text{ (6 ft x } \pi) \; x \; 10 \text{ ft} = 5{,}202 \text{ kips}$$

Rock coring and local experience indicate the rock mass below the socket tip is competent; therefore, unit base resistance can be estimated using Equation 10-23 with $N^*_{cr} = 2.5$:

$$q_{BN} = N^*_{cr} \, q_u = 2.5 \text{ (360 ksf)} = 900 \text{ ksf}$$

Nominal geotechnical base resistance:

$$R_{BN} = \pi/4 \text{ (B)}^2 \; x \; q_{BN} = \pi/4 \text{ (6 ft)}^2 \; x \; 900 \text{ ksf} = 25{,}447 \text{ kips}$$

Factored geotechnical base resistance:

$$\varphi R_{BN} = 0.50 \text{ (25,447 kips)} = 12{,}723 \text{ kips}$$

Check of factored structural bearing resistance (see Chapter 12, Equation 12-7):

$$P_r = \varphi P_n = 0.70 \; [0.85 \, f_c' \, A_1 \, m] \quad = 0.70 \; [0.85 \text{ (576 ksf) } \pi/4 \text{ (6 ft)}^2 \; x \; 2] \qquad \text{(AASHTO 5.6.5-2)}$$

$$P_r = 19{,}380 \text{ kips} > 12{,}723 \text{ kips}$$

The factored geotechnical base resistance governs and is 12,723 kips.

10.3.6 Evaluate Trial Design for Strength Limit States (Step 6)

This step involves application of resistance factors to the nominal resistances determined in Step 5, followed by evaluation of each strength limit state established in Step 2, for each loading mode established in Step 3, and for each trial design (Step 4), to satisfy the strength limit state condition expressed by Equation 10-1:

$$\sum \eta_i \gamma_i \, F_i \leq \sum \varphi_i \, R_i \qquad \text{10-1}$$

Recommended resistance factors for use with nominal resistances calculated by the methods described above are presented in Table 8-4. Note that when compressive resistances are determined on the basis of one or more static load tests, AASHTO (2017a) allows for a resistance factor value as high as 0.70. The actual value selected is a matter of engineering judgment and should account for the number of load tests conducted and the level of subsurface variability, as discussed in Chapter 13.

Illustrative Example 10-5 demonstrates evaluation of a strength limit state check (Step 6).

Illustrative Example 10-5. Perform Strength Limit State Check for Axial Design

For the trial design presented in Illustrative Example 10-1 and depicted in Figure 10-5, the nominal resistances were calculated in Illustrative Examples 10-1, 10-2 and 10-3. Information provided by the structural engineer includes a governing factored axial load demand corresponding to the Strength I limit state load combination:

$$\Sigma \gamma Q = 766 \text{ kips}$$

Based on the calculated resistances, determine whether the trial design satisfies the LRFD criterion.

From Table 8-4 the following resistance factors are applicable:

Component of Resistance	Geomaterial	Resistance Factor, φ
Side resistance in compression	Cohesionless soil	0.55
	Cohesive soil	0.45
Base resistance	Cohesive soil	0.40

By Equation 10-2 the summation of factored resistances consists of the side resistance provided by each of the three layers plus the base resistance, or:

$$\sum \varphi_i R_i = \sum_{i=1}^{n} \varphi_{S,i} \; R_{SN,i} + \varphi_B R_{BN} \qquad 10\text{-}2$$

The table below tabulates each component of nominal resistance, as calculated in previous examples, and multiplies each by the appropriate resistance factor. The factored components are summed to obtain the factored axial resistance of 890.1 kips.

Layer	Geomaterial	Nominal Resistance (kips)	Resistance Factor	Factored Resistance (kips)
1	Cohesionless	$R_{SN,1}$ = 197.2	0.55	108.5
2	Cohesionless	$R_{SN,2}$ = 635.6	0.55	349.6
3	Cohesive	$R_{SN,3}$ = 479.7	0.45	215.9
3	Cohesive	$R_{BN,3}$ = 453.6	0.40	181.4
				$\Sigma\varphi_i R_i$ = 855.4 kips

A limit state check consists of demonstrating that the factored axial demand does not exceed the summation of factored axial resistance (Equation 10-1), or:

$$766 \text{ kips} < 855.4 \text{ kips}$$

The limit state check can also be expressed in terms of the (factored) capacity to demand ratio, or C/D. A C/D value of 1 or greater satisfies the LRFD criterion. In this example C/D is equal to 855.4/766 = 1.12 > 1.0, demonstrating the trial design satisfies the LRFD criterion for axial loading for the Strength I limit state.

10.3.7 Evaluate Trial Design for Service Limit States (Step 7)

Foundation settlement criteria are based on considerations that account for the function and type of structure, anticipated service life, and consequences of unacceptable movements on structural performance. Several studies have been conducted on tolerable settlement for bridges (Moulton et al., 1985, Barker et al., 1991) and these have led to some simple rule-of thumb criteria. For example, a tolerable angular distortion between adjacent foundations is often taken as 0.008 radians in simple spans and 0.004 radians in continuous spans. However, other factors may be taken into account, such as foundation costs, rideability, deck cracking, aesthetics, and others. Therefore, movement criteria are established on a project-specific basis and the foundation designer must work closely with the bridge structural engineer to establish settlement limits for each structure. According to a survey of transportation agencies reported by Paikowsky et al. (2004) tolerable settlement for bridge foundations typically ranges from 0.25 to 2.0 inches.

If expressed in terms of the LRFD criterion (*i.e.*, in terms of force, not deformation), the serviceability limit state is defined by the axial resistance that corresponds to the tolerable axial deformation, $\delta_{service}$. Once the tolerable axial deformation has been established, the trial design is analyzed for axial load-deformation behavior to establish the axial resistance corresponding to the tolerable deformation, which is then evaluated against the LRFD criterion:

$$\sum \eta_i \gamma_i F_i \leq \sum \varphi_i R_i \qquad 10\text{-}30$$

Resistances (R_i) on the right side of Equation 10-1 are the values corresponding to the tolerable axial deformation and the resistance factors (φ_i) are *serviceability* values, currently taken as unity (1.0) in the AASHTO Specifications (AASHTO, 2017a). The load factors (γ_i) on the left side of Equation 10-1 are for serviceability limit states, also currently taken equal to unity. The analysis required to evaluate a service limit state involves determination of the axial load that results in a specified value of axial deformation. Three methods are presented here. The first method is amenable to hand calculations and is based on observations from field load tests. This method is useful for making settlement estimates for single drilled shafts in soil during preliminary design, or to determine that settlement will likely not be critical to the performance of the structure. The second method is based on numerical simulation of axial load transfer and requires an estimate of the load transfer function for each geomaterial layer through which the shaft derives its resistance. This method is normally executed with the use of computer software and is recommended for design of shafts in layered soil and rock profiles, for planning load tests, and in cases where settlement can potentially govern the design. A third method is presented for predicting settlement of shafts bearing on or socketed into rock or other very dense geomaterials. This method is based on approximations derived from elasto-plastic analytical solutions and is amenable to spreadsheet calculations. The first method is presented below followed by an illustrative example. A general overview is then presented for the second and third methods, with further details given in Appendix C.

Cohesionless and Cohesive Soils, Simple Method

A simple method for estimating the load-deformation behavior of drilled shafts in axial compression is based on modeling the average behavior observed from field load testing. For this purpose, load test data are presented in normalized form, in which axial force is normalized by the nominal resistance (summation of predicted side and base resistances) and plotted against displacement (δ) normalized by shaft diameter, B. A normalized load-displacement curve representing the average observed behavior can then be used as a very approximate guide for estimating axial displacements under service load conditions. The principal limitation of this approach is that "average" curves developed in this manner are based on data that typically exhibit large degrees of variability (scatter). Axial deformation predicted from the resulting curve does not

represent a rigorous analysis based on site-specific soil properties, but rather a crude first-order estimate considering average behavior. The advantage of this approach lies in its simplicity and ease of use. It is most suitable for conducting preliminary analyses of trial designs or as a means of conducting service limit state checks to establish whether axial deformation is likely to be of significant concern.

In a previous version of this manual (O'Neill and Reese, 1999), normalized load-deformation curves, referred to as load transfer curves, were presented separately for side resistance and base resistance. Analysis of axial deformation required iterating between the two figures. The original O'Neill and Reese load-transfer curves are reproduced in Appendix C for historical reference. The curves presented herein, based on Chen and Kulhawy (2002), combine the side and base resistances into single curves representing both components of axial resistance. The model was developed primarily as a way to provide consistent interpretation of load test results. The data used to develop the curves include the load test data used by O'Neill and Reese (1999) plus additional load test data collected by the method's developers. The resulting curves are easier to use and incorporate a larger amount of load test data.

Figure 10-10 presents normalized load-displacement curves for drilled shafts in soil under axial compression. Two curves are shown to differentiate between undrained loading in cohesive soils (dashed line) and drained loading in cohesionless soils (solid line). The figure is adapted from Chen and Kulhawy (2002) based on analysis of a worldwide database of compression load tests on straight-sided shafts. The analysis for cohesionless soils is based on 46 data points while the curve for cohesive soils is based on 56 data points. In Figure 10-10 each load-displacement curve is defined by the following three regions:

1. From the origin to a normalized displacement (δ/B) of 0.4% the behavior is linear. The upper end of this region corresponds to a normalized axial compressive force of 50%.
2. Between normalized displacements of 0.4% and 4.0% the response is nonlinear as the shaft yields. The normalized axial force at $\delta/B = 4.0\%$ is considered to be the interpreted failure load or "failure threshold".
3. Beyond the failure threshold is the final linear region. For cohesive soils this final region is represented by a constant resistance corresponding to a normalized axial force of 100%. For cohesionless soils the shaft resistance continues to increase in response to the large displacement required to mobilize base resistance.

In developing the average curves shown in Figure 10-10, Chen and Kulhawy (2002) considered the axial force acting on the shaft to consist of the force applied to the top of the shaft plus the weight of the shaft (W). For cohesive soils, the failure threshold, or nominal axial resistance, corresponds to mobilization of the full available side resistance (R_{SN}) plus the full available base resistance (R_{BN}). In cohesionless soils, the failure threshold is the force corresponding to mobilization of the full side resistance (R_{SN}) plus the base resistance corresponding to $\delta/B = 4.0\%$ ($R_{BN\ 4.0\%}$). The reader will note that nominal base resistance in cohesionless soils is calculated in this manual using the empirical correlation given by Equation 10-14, in terms of N-value. That relationship was developed using base resistance corresponding to 5 percent normalized displacement. To use the cohesionless soil curve depicted in Figure 10-10, therefore, the base resistance calculated by Equation 10-14 must be corrected to a normalized displacement of 4 percent. This requires multiplying the base resistance from Equation 10-14 by a factor of 0.71. This number is simply the ratio of tip resistances mobilized at 4 percent and 5 percent normalized displacements, respectively.

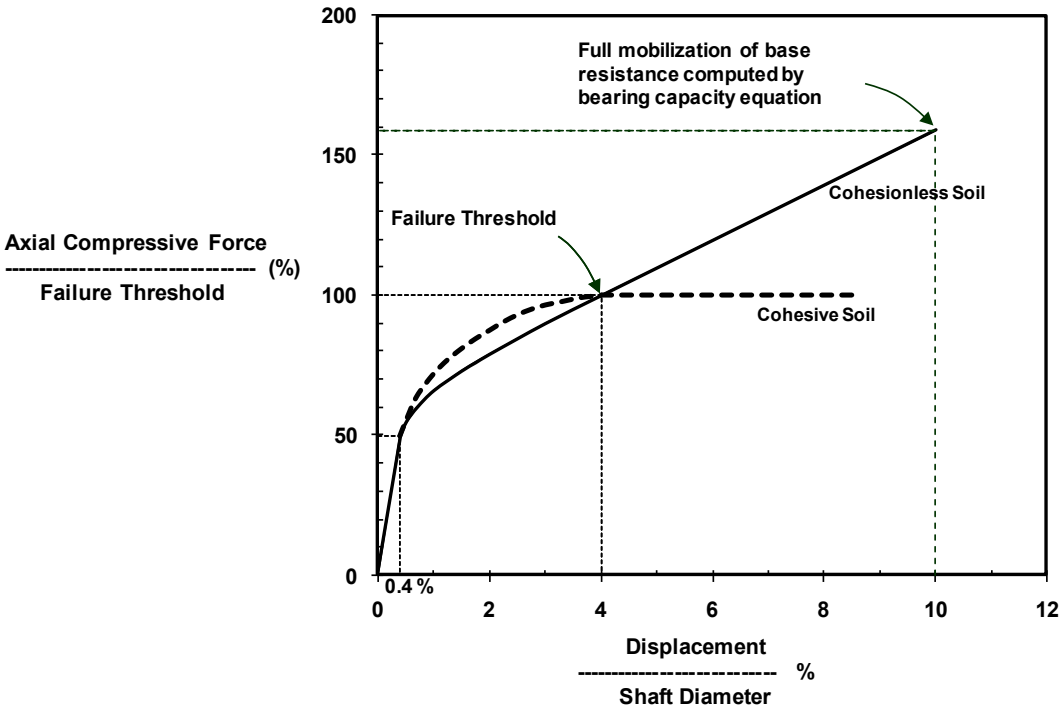


Figure 10-10 Normalized Load-Displacement Curve, Drilled Shaft in Axial Compression (Adapted from Chen and Kulhawy, 2002)

Additional limitations of Figure 10-10 are that the data used in its development include drilled shafts with diameters ranging from 1 ft to 6.5 ft, depths ranging from 16 ft to 200 ft, and depth to diameter ratios ranging from 5.8 to 56.4. Use of the curves shown in the figure should be limited to drilled shafts with geometries falling within these ranges. In the analysis of field load tests used to develop Figure 10-10, shaft displacements were taken as the measured butt displacement and therefore incorporate elastic compression of the concrete shaft.

Figure 10-10 can be used to make a first-order estimate of the settlement of straight-sided shafts in soils as follows:

1. The failure threshold is computed as the sum of nominal side and base resistances calculated in Step 11-6:

$$\text{failure threshold} = R_{SN} + R_{BN} = Q_c + W \qquad 10\text{-}31$$

   in which R_{SN} and R_{BN} are nominal side and base resistances, respectively, Q_c is the compressive force applied at the top of the shaft, and W = weight of the drilled shaft. The weight term is the effective weight of the foundation, given by the total weight above the water table and the buoyant weight below the water table.
2. In Equation 10-30, the nominal resistances R_{SN} and R_{BN} for shafts in cohesive soils are calculated using the equations presented in Section 10.3.5.2. For cohesionless soils, R_{SN} is calculated using the beta method as presented in Section 10.3.5.1, while the base resistance is calculated by multiplying by 0.71 the value obtained by the use of Equation 10-14, as explained above (corresponding to nominal base resistance at 4.0% normalized settlement).

3. Divide the tolerable settlement, $\delta_{service}$, by the shaft diameter to obtain the normalized displacement ($\delta_{service}/B$) expressed as a percentage.
4. Enter Figure 10-10 with the normalized displacement from Step 2 and, using the appropriate curve, determine the corresponding normalized axial force.
5. Divide the normalized force by 100% and multiply by the "failure threshold" load to obtain the Axial Compressive Force. This force = $R_{service} + W$, where $R_{service}$ = applied axial compressive force corresponding to the tolerable settlement. Subtract the shaft weight from the Axial Compressive Force to obtain $R_{service}$.
6. Substitute the value $R_{service}$ into the right-hand side of Equation 10-1 to evaluate the service limit state criterion (*i.e.*, a service limit state check).

The methods used by Chen and Kulhawy (2002) to calculate the other components of resistance are: β-method for side resistance in cohesionless soils; α-method for side resistance in cohesive soils; and bearing capacity theory for base resistance in cohesive soils. These are the methods presented in Section 13.3.3. The figure is suitable for making simple, approximate calculations of axial load-deformation behavior, keeping in mind that both curves represent average behavior and are not to be considered as rigorous analytical predictions.

Figure 10-10 also provides a tool for making preliminary estimates of load-settlement behavior for shafts in mixed soil profiles. In calculating the failure threshold, the nominal base resistance must account for the soil type providing base resistance. This will also determine which of the two curves (cohesionless soil or cohesive soil) to use for evaluation of settlement.

The following example is presented to illustrate the application of this method to a trial drilled shaft design with its tip bearing in cohesive soil.

Illustrative Example 10-6. Service Limit State Evaluation of Trial Shaft

In this example, the trial drilled shaft design introduced as Example 10-1 and used to illustrate calculations of side and base resistance in Examples 10-1, 10-2, 10-3 and 10-5, is checked for Service I Limit State loading conditions. Structural modeling of the superstructure, incorporating the trial foundation design, has been conducted and the factored Service I axial force effects and tolerable settlement provided by the structural engineer are as follows:

Service I Limit State: tolerable axial settlement = 1.5 inch
under nominal compressive load = 575 kips

Utilize Figure 10-10 as a first-order approximation of the axial load transfer behavior.

Service Limit State Calculations: using the step-by-step procedure described above:

1. By Equation 10-30: failure threshold = $R_{SN} + R_{BN} = Q_c + W$

where, from previous examples: $R_{SN} = 197.2 + 635.6 + 479.7 = 1{,}312.5$ kips

$R_{BN} = 453.6$ kips

$$W = \frac{\pi}{4}(5\text{ ft})^2[15\text{ ft}(150\text{pcf}) + 45\text{ft}(150 - 62.4\text{pcf})]$$

= 121,580 lb = 121.6 kips

Failure threshold = 1,312.5 + 453.6 = 1,766.1 kips

2. normalized tolerable displacement: $\frac{\delta_{tolerable}}{B} = \frac{1.5\,inch}{60\,inch}100\% = 2.5\%$

3. entering Figure 13-10 with δ/B = 2.5%, normalized axial load = 93% (cohesive soil curve)

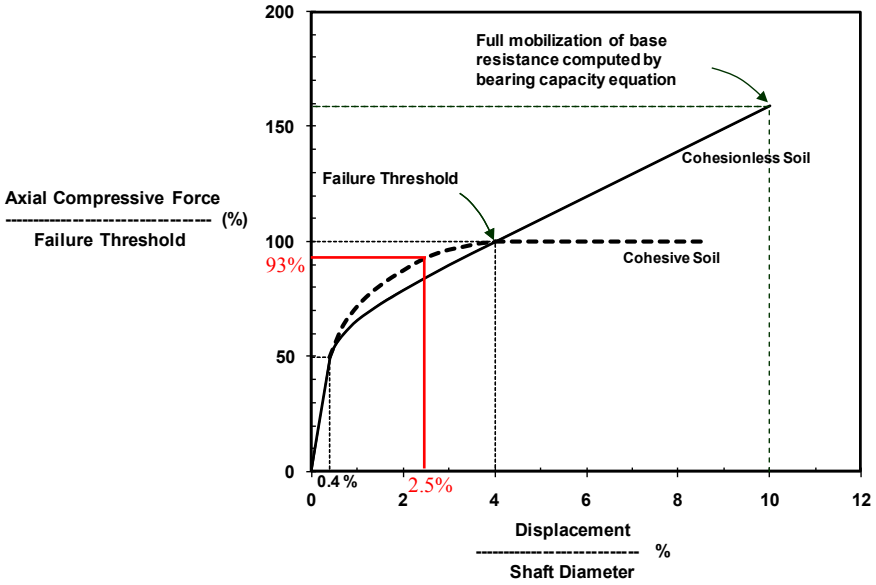


Figure 10-11 Analysis of Service Limit State Displacement for Example 10-6

4. $R_{service} + W = 0.93\ (1{,}766.1\ kips) = 1{,}642.5\ kips$

5. $R_{service} = 1{,}642.5\ kips - 121.6\ kips = 1{,}521\ kips$

6. Service limit state check: $\sum \eta_i \gamma_i F_i \le \sum \varphi_i R_i$

575 kips < 1,521 kips

The Service I Limit State is easily satisfied by the trial design.

Finally, the designer may wish to estimate the expected settlement under the service load. This can be done using Figure 13-10, as follows:

Axial compressive force = Service I axial force + W = 575 + 121.6 = 696.6 kips

Normalized axial force = axial compressive force / failure threshold = 696.6 kips / 1,766 kips = 0.39
= 39 percent

Entering Figure 10-10 with normalized axial force = 39%:

Normalized displacement = 0.40% x (39/50) = 0.31%

Settlement = normalized displacement times shaft diameter = 0.0031 x 60 inches = **0.19 inch**

Expected settlement under the Service I axial load is less than 0.2 inches and well within the tolerable settlement of 1.5 inch. No further analysis would be needed for this limit state load case.

Numerical Simulation of Load Transfer

For drilled shafts in layered subsurface profiles and/or where a large number of potential loading cases and trial designs are to be analyzed, numerical simulation of axial load-deformation response provides a practical design tool. In this approach, the drilled shaft and surrounding geomaterials are idealized as a set of linear and nonlinear springs, as illustrated in Figure 10-12. The concrete shaft is modeled by a linear spring to represent its elastic compression or extension in response to axial load. The stiffness of the shaft (spring) is normally constant with depth, although it can be varied. Each geomaterial layer is replaced by a nonlinear spring representing the axial load transfer by development of side resistance. This relationship is expressed in terms of the shear stress developed along the interface (t) and the relative movement between the shaft and geomaterial (z). The mechanism representing this t-z curve consists of a cantilever spring and friction block, as shown in Figure 10-12b. A single nonlinear spring is also applied to represent the base load transfer in terms of unit base resistance (q) versus base displacement (w_b). The t-z and q-w_b curves, shown in Figure 10-12c, can be represented by analytical functions.

Based on the model shown in Figure 10-12, a differential equation can be written that satisfies equilibrium and compatibility between the load and deformation at any point along the shaft. Additional details, including the governing differential equation, are presented in Appendix D. The governing differential equation with appropriate mathematical functions to model the t-z and q-w_b curves and boundary conditions (axial load) can then be solved numerically, usually through a finite difference scheme, to obtain a simulated load-deformation curve for the axially loaded drilled shaft. Commercially available computer software is generally used for this purpose.

The accuracy of computer-generated solutions to this problem depends entirely on the accuracy of the t-z and q–w_b curves (load transfer curves) used as input. When using commercial software, the user normally has the option of inputting load transfer curves developed specifically for the site conditions, or using load transfer curves generated internally by the program based on soil type (cohesive or cohesionless) and nominal unit side resistance (f_{NS}) for each layer. It is emphasized that research has not yet advanced to the point that load transfer curves can be predicted with confidence for all conditions encountered in practice. Additionally, the load transfer behavior of some geomaterials may be sensitive to construction practices. On major projects, it is advisable to conduct one or more field load tests during the design phase, with instrumentation to measure load transfer curves. The computer program can then be calibrated to achieve agreement with the measured curves. The calibrated, site-specific curves can then be used to analyze trial designs, which may vary from the test shaft in geometry and stiffness.

This approach assumes that the construction methods and tools used to install the production shafts will be similar to those used for the design-phase load test program; other construction methods could result in a different response to shaft loading.

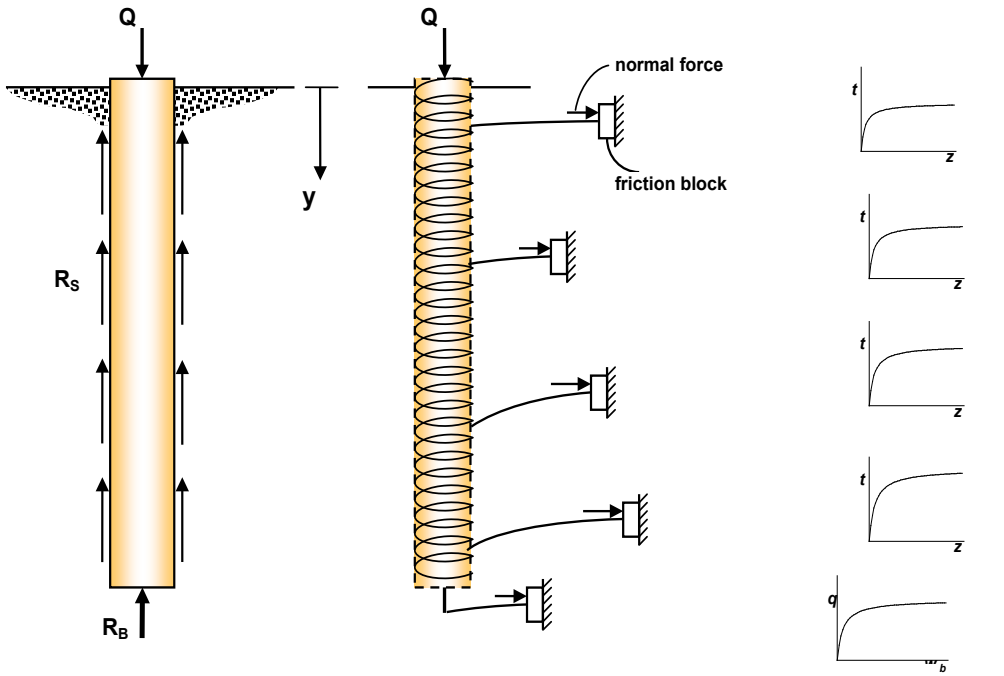


Figure 10-12 Mechanistic Model of Axially Loaded Drilled Shaft (after Reese et al., 2006)

Approximate Closed-Form Solutions

Several researchers have developed approximate solutions to the load-displacement behavior of drilled shafts embedded in stiff materials, such as rock or very dense geomaterials, that agree well with more advanced analytical models based on theories of elasticity and plasticity, finite element analysis, and detailed back-analysis of field load tests. The resulting closed-form solutions offer the advantage of being easily implemented in a spreadsheet format, thus providing a convenient desktop tool for the foundation designer. The equations are given in Appendix C. A brief overview of the available methods and their applicability is given below.

The basic problem is depicted in Figure 10-13 and involves predicting the relationship between an axial compression load (Q_c) applied to the top of a socketed drilled 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 D and diameter B has Young's modulus E_c and Poisson's ratio, ν_c. The rock mass surrounding the cylinder is homogeneous with Young's modulus E_r and Poisson's ratio ν_r while the rock mass beneath the base of the shaft has Young's modulus E_b and Poisson's ratio ν_b. The 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 $\sigma_b = 4Q_c/(\pi B^2)$.

Approximate closed-form solutions are given in Appendix D for the cases of:

1. Rock socketed shafts, by Carter and Kulhawy (1988);
2. Shafts in dense cohesionless soils with SPT N-values > 50, by Mayne and Harris (1993), and

The required input parameters for each method are discussed in Appendix C.

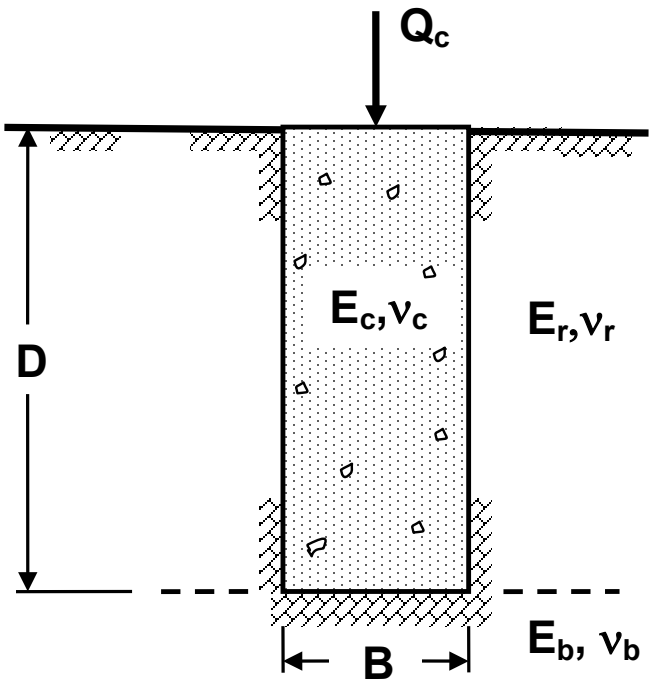


Figure 10-13 Axially Loaded Shaft in Rock or Very Hard Geomaterial, Elastic Analysis

10.3.8 Final Trial Design for Axial Compression

The step by step procedure presented above is implemented through an iterative process. Trial designs (Step 4) are evaluated for LRFD strength limit states (Steps 5 and 6) and service limit states (Step 7). If a trial design fails to satisfy one or more of the required limit states, or if the trial design greatly exceeds all of the required limit states and a more economical design is possible, dimensions of the trial design are modified and re-analyzed through Steps 5, 6 and 7. This process is continued until all applicable limit states for axial compression satisfy the LRFD criterion (Equation 10-1). For compression, this portion of design for axial loading constitutes completion of the step-by-step procedure depicted in Figure 10-2, which constitutes Block 11 of the overall design process outlined in Chapter 8.

10.4 DESIGN FOR UPLIFT

Loads acting on deep foundations supporting transportation structures may include uplift. A typical uplift case occurs when a foundation for a bridge pier consists of multiple shafts beneath a footing or pile cap. Structural force effects transmitted to the top of footing may consist of combined axial compression, Q_N and bending moment, M_N, as shown in Figure 10-14. The moment is resisted by the force couple provided by uplift resistance of drilled shafts on one side of the footing and compressive resistance provided by shafts on the opposite side. In Figure 10-14, the clockwise moment transmitted to the footing results in uplift of shafts on the left side and compression of shafts on the right side. When the combined effects of axial compression and moment acting on the entire foundation are considered for an individual shaft, the net force may be uplift.

Other conditions leading to uplift include construction loads induced during erection of concrete segmental girder bridges, vehicular loading on adjacent spans of continuous girders, extreme event loads such as

seismic and impact, and others. In all of these cases, the drilled shaft must be evaluated for applicable limit states under the uplift force effects.

As illustrated in Figure 10-15, a drilled shaft loaded in uplift is resisted by the weight of the shaft and the side resistance that develops over the cylindrical surface at the interface between the concrete shaft and the geomaterial layers along the sides. The design process follows the same general steps presented in Section 10.2 for axial compression, as summarized in the flowchart of Figure 10-2. For each design zone an idealized profile is developed, as illustrated in Figure 10-15. Each layer within the zone is assigned a layer number *i*, thickness (Δz_i), and geomaterial type.

AASHTO Specifications (2017a) require the "uplift resistance of a single straight-sided drilled shaft to be estimated in a manner similar to that for determining side resistance for drilled shafts in compression". Therefore, nominal unit side resistances are computed by the same methods presented in Section 10.3.5 for axial compression loading.

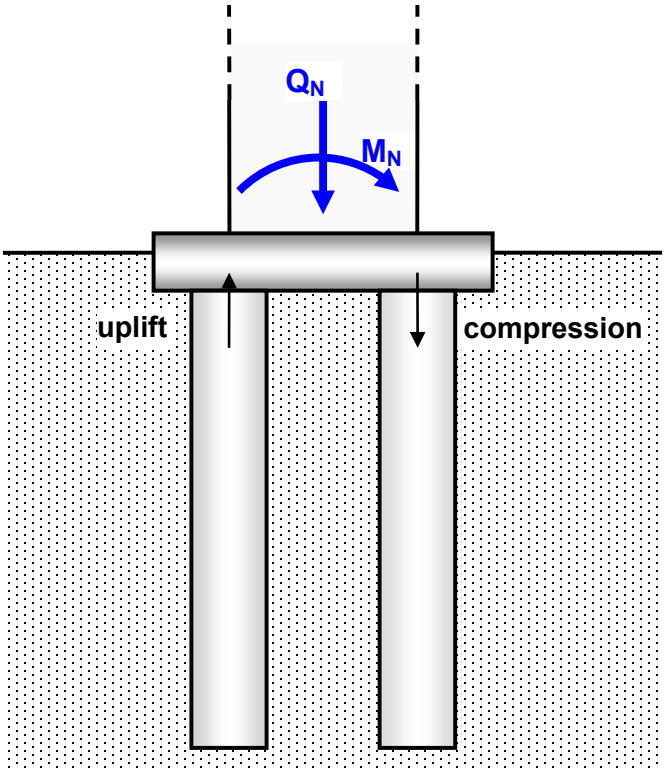


Figure 10-14 Typical Loading Combination Resulting in Uplift

Resistance to uplift may develop at the base of a drilled shaft under short-term loading, as negative porewater pressure (suction) develops in response to the void created between the base of the shaft and underlying ground. In cohesionless soils, any suction would dissipate rapidly, but in cohesive soils (clay), tip suction could be substantial for the short-term undrained condition. However, prediction of tip suction for design purposes is not sufficiently reliable and therefore no attempt will be made to incorporate base resistance into design for uplift loading in this manual.

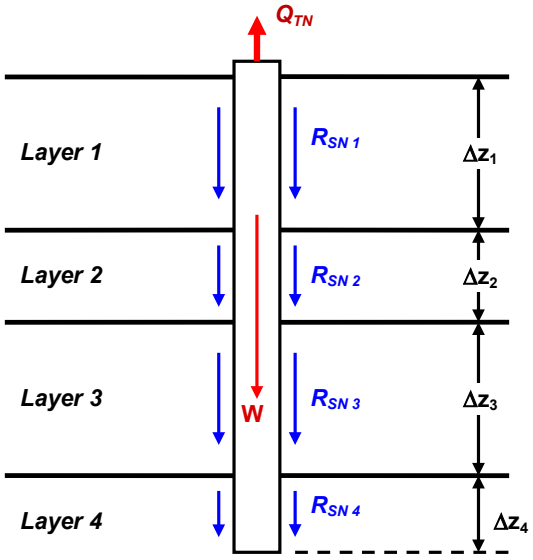


Figure 10-15 Forces and Idealized Geomaterial Layering for Computation of Uplift Resistance

For design purposes the weight of the drilled shaft is treated as a (negative) force effect on the left side of Equation 10-1, rather than as a component of resistance. Accordingly, resistance to axial uplift loading consists only of side resistance and evaluation of the LRFD strength limit state is therefore given by:

$$\sum \varphi_i R_i = \sum_{i=1}^{n} \varphi_{S,i} R_{SN,i} \qquad 10\text{-}32$$

where:

$R_{SN,i}$ = nominal side resistance for layer i,
$\varphi_{S,i}$ = resistance factor for layer i, and
n = number of layers providing side resistance.

Resistance factors for uplift (side resistance) are presented in Table 8-4. The uplift values are less by 0.10 than the equivalent resistance factors for compression, as discussed by Allen (2005) and adopted in the AASHTO Specifications (2017a). This approach is considered an interim procedure pending completion of reliability-based calibration of uplift resistance factors.

Where side resistance values are established on the basis of one or more static uplift load tests conducted at the site of the proposed bridge or other structure, current AASHTO Specifications (2017a) allow a resistance factor of 0.60, for all geomaterials. Considerable judgment should be exercised in applying this value, based on the degree to which conditions at the site of production shafts match the conditions of the load test. Factors, such as the variability in stratigraphy, soil and rock properties, shaft dimensions, and construction method, similar to those discussed for drilled shafts in compression should be considered to justify direct application of the load test results to production shafts. Otherwise, a lower resistance factor should be applied. The degree to which the resistance factor is lowered is based on engineering judgment, and should consider foundation redundancy and the consequences of upward displacement of the foundation. If the uplift load is caused by an extreme event, such as earthquake, ice loading, or vessel impact, the uplift resistance factor value can be taken as 0.80.

The trial design is then evaluated for each applicable strength limit state, using the general form of the LRFD criteria as given by Equation 10-1.

10.5 DESIGN FOR SCOUR

10.5.1 Background and Definitions

Bridge scour is the loss of soil by erosion due to flowing water around bridge supports. Total scour at a highway crossing is comprised of three components:

1. Long-term aggradation and degradation of the river bed
2. Contraction scour at the bridge
3. Local scour at piers or abutments

Aggradation is the general and progressive buildup of the longitudinal profile of a channel bed due to sediment deposition, while degradation is the general and progressive lowering of the channel bed due to erosion; these are processes predicted to occur without the presence of the bridge. Contraction is defined as constriction (narrowing) of the channel as a result of bridge construction or other factors. Contraction scour is the resulting removal of material from the bed and banks across all or most of the channel width in response to the increased flow velocity resulting from contraction of the channel.

Local scour is the erosion of material around obstacles to the water flow. It is caused by the localized acceleration of flow and the resulting vortices induced by the obstructions. Local scour occurs at bridge piers and abutments.
Total scour is the sum of long-term degradation, contraction scour, and local scour. Figure 10-16 illustrates the components of scour that may affect bridges supported on deep foundations. Effects of scour that must be taken into account for drilled shaft design include: (1) changes in subsurface stress, (2) reduced embedment and therefore changes in geotechnical resistances, and (3) possible changes in the response of the structure and the resulting foundation force effects (loading).

Channel migration, which is not identified as a specific component of scour, is defined as the lateral migration of an alluvial river channel across its floodplain. This process is mainly driven by the combination of scour (erosion) of the bank combined with point bar deposition over time, and can have the same effect on foundations, *i.e.*, either removal or buildup of soil at pier and abutment locations

10.5.2 Design Philosophy for Scour

The FHWA document "Evaluating Scour at Bridges" (Arneson et al., 2012), commonly known as HEC-18, presents practical methods for analysis of scour, and general procedures for design of bridges susceptible to scour. The following paragraphs summarize the general approach to bridge scour as presented in HEC-18.

Foundations for new bridges should be designed to withstand the effects of scour caused by the hydraulic conditions from floods larger than the design flood. The *hydraulic design flood* is selected by the Owner based on a risk-based approach that factors in the importance of the structure, safety, reliability, and consequences of failure. Once the frequency of the hydraulic design flood is established, the minimum *scour design flood* and the *scour design check flood* are selected in accordance with Table 10-4. In Table 10-4, Q_D refers to the hydraulic event (flood) with a return interval equal to D in years (*e.g.*, Q_{25} is the flood

with a return interval of 25 years). Both of the scour floods are larger than the hydraulic design flood. This approach is based on the philosophy that a bridge must be designed to a higher level for scour than for the hydraulic design because if the hydraulic design flood is exceeded, of which there is a reasonably high likelihood during the service life of the bridge, then a greater amount of scour will occur which could lead to bridge failure. Also, designing for a higher level of scour than the hydraulic design flood ensures a level of redundancy after the hydraulic event occurs. Bridges are to be designed without failing for the worst conditions resulting from the scour design flood. Bridge foundations should be checked to ensure that they will not fail due to scour resulting from the occurrence of the scour design *check* flood. The scour check flood is an extreme event; the scour design flood is not an extreme event. Scour due to the scour design flood is considered for all strength and service limit states for which the bridge is designed.

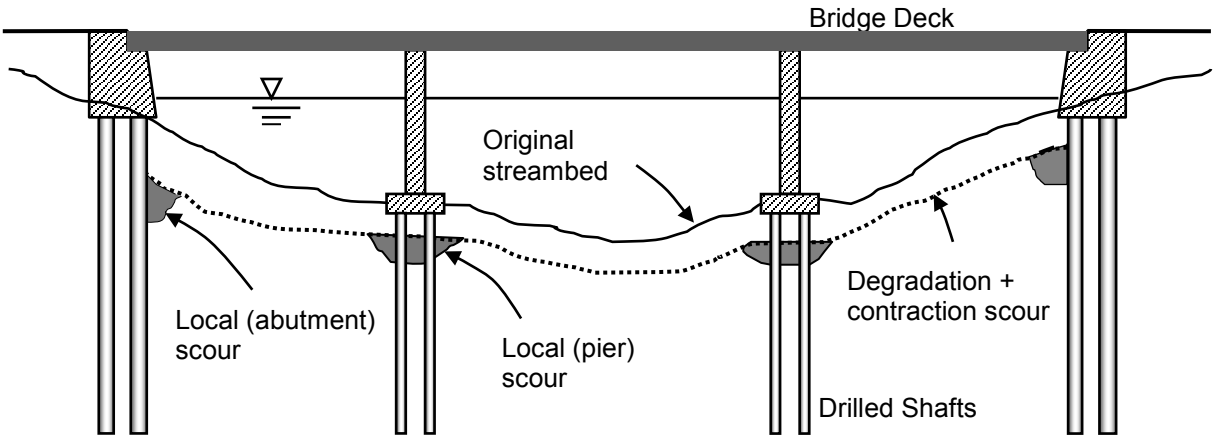


Figure 10-16 Components of Scour Affecting Bridge Supports on Deep Foundations

Table 10-4 FLOOD FREQUENCIES FOR HYDRAULIC DESIGN, SCOUR DESIGN, AND SCOUR DESIGN CHECK FLOODS (based on Table 2.1, HEC-18)

Hydraulic Design Flood Frequency, Q_D	Scour Design Flood Frequency, Q_D	Scour Design Check Flood Frequency, Q_D
Q_{10}	Q_{25}	Q_{50}
Q_{25}	Q_{50}	Q_{100}
Q_{50}	Q_{100}	Q_{200}
Q_{100}	Q_{200}	Q_{500}

The approach to design for scour is based on the following concepts:

1. The foundation should be designed by an interdisciplinary team of engineers with expertise in hydraulic, geotechnical, and structural design.
2. Hydraulic studies of bridge sites are a necessary part of a bridge design. These studies should address (a) the sizing of the bridge waterway opening to minimize adverse impacts to upstream and downstream landowners and to provide safe foundations. and (b) calculations of scour at the bridge foundations. The scope of the hydraulic analysis should be commensurate with the complexity of the situation, the importance of the highway, and consequences of failure.
3. Consideration must be given to the limitations and gaps in existing knowledge when using currently available formulas for estimating scour. The interdisciplinary team needs to apply engineering judgment

in comparing results obtained from scour computations with available hydrologic and hydraulic data and conditions at the site to achieve a reasonable and prudent design. Such data should include:

a. Performance of existing structures during past floods
b. Effects of regulation and control of flood discharges
c. Hydrologic characteristics and flood history of the stream and similar streams
d. Whether the bridge has redundant structural elements
e. Historical trends for channel migration

4. It must be recognized that occasional damage to highway approaches from rare floods can be repaired quickly to restore traffic service. On the other hand, a bridge which collapses or suffers major structural damage from scour can create safety hazards to motorists as well as significant social impacts and economic losses over a longer period of time. Aside from the costs to the DOTs of replacing or repairing the bridge and constructing and maintaining detours, there can be significant costs to communities or entire regions due to additional detour travel time, inconvenience, and lost business opportunities. Therefore, a higher hydraulic standard is warranted for the design of bridge foundations to resist scour than is usually required for sizing of the bridge approaches.

10.5.3 Analysis and Prediction of Scour

The overall process for addressing scour at bridge sites involves the following three stages of assessment and analysis:

1) A stream stability and geomorphic assessment is conducted in accordance with the publication "Stream Stability at Highway Structures" also denoted as HEC-20 (Lagasse et al., 2012).
2) A scour analysis is conducted in accordance with HEC-18 (Arneson et al., 2012). This stage also involves foundation design and is most relevant to drilled shaft design. A flow chart of the scour analysis procedure is shown in Figure 10-17. For existing bridges determined to be scour critical, a new bridge design can be developed or the process can proceed to Stage 3.
3) A plan of action is developed to undertake bridge scour and stream instability countermeasures. This final stage does not apply to new designs and is therefore not relevant to drilled shaft design.

Figure 10-17 also indicates the interaction that is required between hydraulic, geotechnical, structural, and civil engineers. The hydrologic and hydraulic analyses typically are conducted by a hydraulic specialist, but with structural and geotechnical input. Geotechnical information required at this stage includes subsurface profiles based on borings and geophysical tests, classification of geomaterials according to the Unified Soil Classification System, and evaluation of the erodibility of subsurface units. Structural input includes the number, location, and geometry of bridge piers and abutments. Civil engineers provide information on the station limits of the approach embankments, and the location and size of any hydraulic structures to be constructed through the embankments. Based on the scour analysis, a scour prism is developed for each pier and abutment. The design team then conducts a multidisciplinary evaluation of the bridge. Geotechnical input to this evaluation requires analysis of strength, service, and extreme event limit states of the deep foundations assuming a subsurface profile that accounts for the scour prism. If the bridge is determined to be stable, the trial design is considered low risk and is acceptable. If not, the design can be modified or measures can be taken to counter the effects of scour, such as lengthening the bridge, increasing bridge spans, and/or changing the number, size and arrangement of drilled shafts at each foundation. Generally, the foundations are designed for the predicted scour, rather than relying on ancillary measures, such as rip-rap armour protection, to reduce or prevent scour at the foundation locations, since the long-term effectiveness of such measures is typically unreliable.

Detailed methods and equations used to determine total scour, consisting of degradation, contraction scour, and local scour, are given in HEC-18 and are not repeated herein. The principal output resulting from scour analysis, and the result that is most relevant to drilled shaft design, is the scour prism, which is a graphical representation of the total volume of material removed by scour.

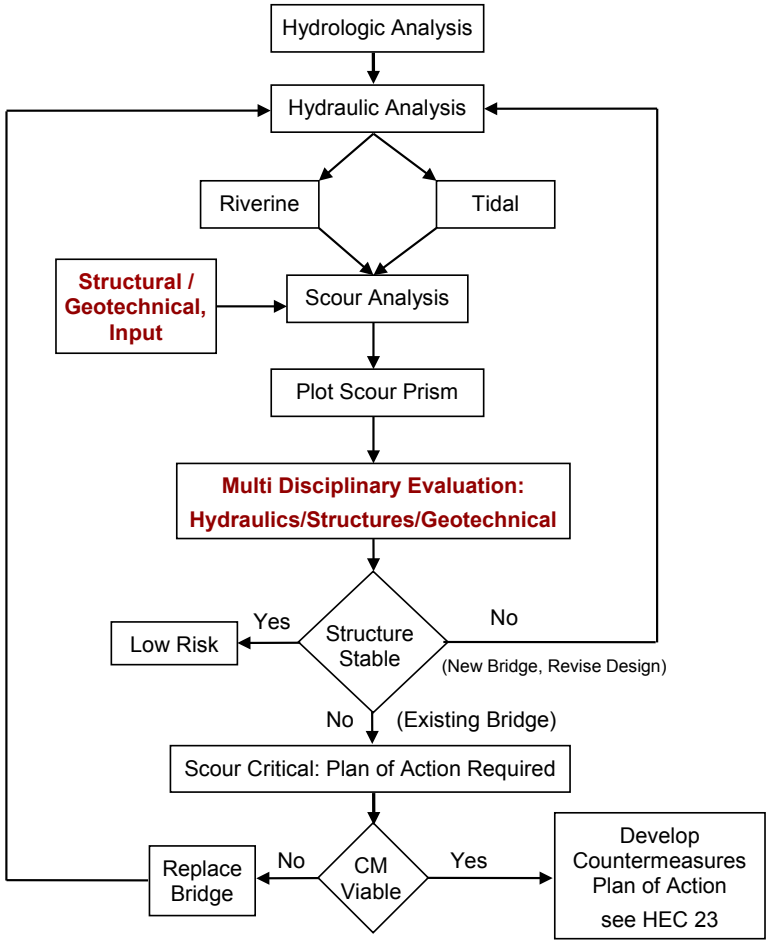


Figure 10-17 Flow Chart for Hydrology, Hydraulics, and Scour Analysis by HEC-18 (Adapted from Arneson et al., 2012)

10.5.4 Application to Drilled Shaft Design

AASHTO Specifications (2017a) require evaluation of bridge foundations for two scour conditions: (1) the *design* flood scour condition is assumed for foundation strength and service limit state evaluations, and (2) the *check* flood scour condition is assumed for extreme limit state evaluation. The commentary under Article 10.5.5.2.1 states:

> "Scour design for the design flood must satisfy the requirement that the factored foundation resistance after scour is greater than the factored load determined with the scoured soil removed. The resistance factors will be those used in the Strength Limit State, without scour."

Therefore, the resistance factors given in Table 8-4 are used for strength limit state analysis of foundations under scour resulting from the scour design flood. Resistance factors given in Table 8-4 for Extreme Event II are applied for evaluation of the Extreme Event II limit state in conjunction with scour resulting from the scour design check flood.

For each strength or service limit state evaluation, all material in the scour prism above the total scour line is assumed to be removed and unavailable for axial or lateral support. Changes in subsurface stress also occur in response to removal of soil, and these stress changes may affect foundation resistances to both axial and lateral loads. Stress changes will vary depending on whether scour occurs over a large area (degradation and contraction scour) or a small area (local scour). Where local scour is limited to a small area around the shaft, the streambed associated with degradation and contraction scour will have a greater influence on stress conditions along the embedded length of the drilled shaft

Figure 10-18 shows a single drilled shaft at a bridge support where scour analysis produces the scour prism as indicated by the shaded area. The original, pre-scour streambed elevation is shown (elevation of point A). The streambed is lowered to the elevation of point B based on the predicted degradation plus contraction scour. A local scour hole is predicted around the pier. The depth of the local scour hole, y_s, is computed using methods prescribed in HEC-18. A top width value (w) of 2.0 times the depth of local scour on each side of the pier is suggested for practical applications (Arneson et al., 2012).

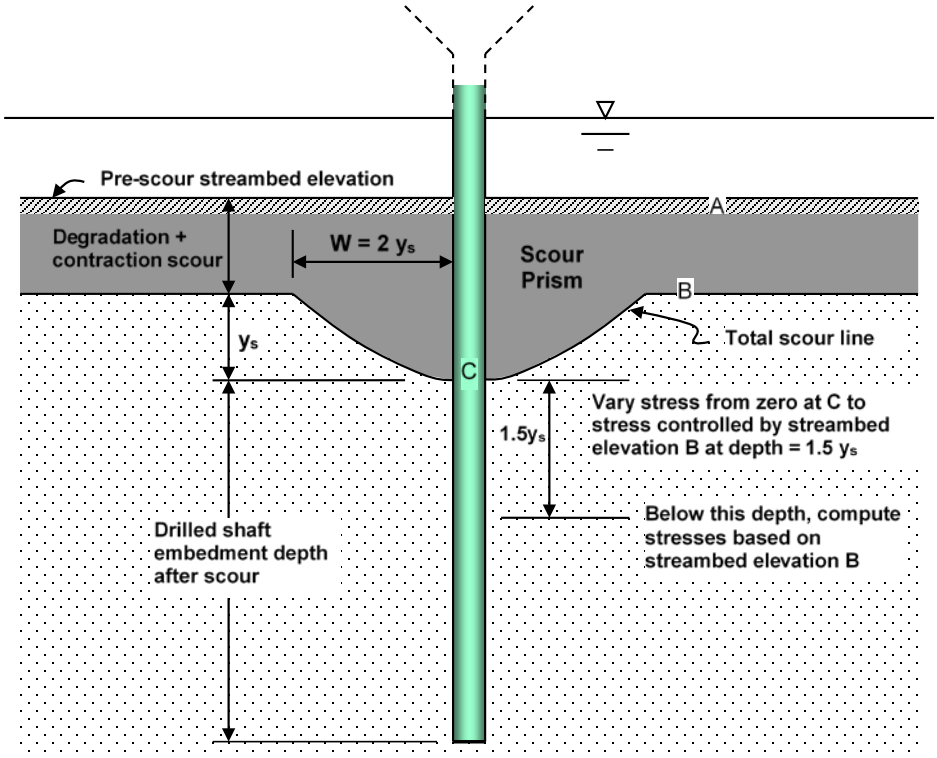


Figure 10-18 Illustration of Scour Prism and Effects on Drilled Shaft

Vertical stress at any depth along the shaft can be estimated as follows. At the top of the embedded length of the shaft (Point C) the vertical stress is equal to zero. At a depth of embedment equal to 1.5 y_s or greater, assume the vertical stress is controlled by the streambed elevation B. Assume a linear variation in vertical stress over the depth 1.5 y_s. In other words, the effect of local scour on stress diminishes linearly over a depth equal to 1.5 times the scour hole depth, y_s.

10.5.5 Effects of Scour on Axial Resistance

Scour conditions are incorporated into analysis of side resistance in cohesionless soils using the beta-method equations presented in Section 10.3.5.1. Note that in the beta-method equation for unit side resistance:

$$f_{SN} = \sigma'_v \beta \qquad 10\text{-}7$$

Since vertical effective stress σ'_v is one of the parameters that determines side resistance, changes in the vertical stress over the portion of the drilled shaft embedded in cohesionless layers can be taken into account as illustrated in Figure 10-18 and described above.

Nominal side resistance in cohesive soils is normally calculated by the α-method (Section 10.3.5.2) In cohesive soils, the undrained shear strength and resulting unit side resistance can be assumed to be unchanged due to changes in effective overburden stress caused by scour. Similarly, the strength and nominal axial resistances provided by rock are assumed to be unchanged due to effects of scour.

Additional issues to be considered by foundation designers in connection with scour include:

- Drilled shaft design may require consideration of column action because of the increase in unsupported shaft length after scour.
- Local scour holes at piers and abutments may overlap in some instances. If local scour holes overlap, the scour is indeterminate and may be deeper. The top width of a local scour hole on each side of the pier ranges from 1.0 to 2.8 times the depth of local scour. A top width value of 2.0 times the depth of local scour on each side of a pier is suggested for practical applications.
- Placing the top of a shaft-supported footing or cap below the streambed at a depth equal to the estimated long term degradation and contraction scour depth will minimize obstruction to flood flows and resulting local scour. Even lower footing elevations may be desirable for shaft supported footings when the drilled shafts could be damaged by erosion from exposure to river or tidal currents. However, in deep water situations, it may be more cost effective to situate the pile cap at or above the mudline or at the waterline, and design the foundation accordingly.
- Stub abutments positioned in the embankment should be founded on drilled shafts or piles extending below the elevation of the thalweg including long term degradation and contraction scour in the bridge waterway to assure structural integrity in the event the thalweg shifts and the bed material around the abutment foundations scours to the thalweg elevation. Thalweg is defined as the lowest elevation of the streambed.
- Estimates of contraction and local scour must consider the foundation cap, if any, and any pier protection system attached to the pier or installed around the pier.

10.6 DOWNDRAG

10.6.1 Occurrence

Downdrag refers to the downward movement of a deep foundation in response to downward-acting forces, or drag loads, transferred to the foundation by surrounding soil that undergoes downward vertical displacement (settlement or slope movement). Load transfer is by shearing stress that develops at the soil-foundation interface. Briaud and Tucker (1997) present an overview of the downdrag problem and note

that downdrag has been reported to result in "extreme movements, differential settlements, and extensive damage to various structures" including highway bridges.

Bridge abutments, shown in Figure 10-19, represent a commonly encountered downdrag situation for transportation projects. A typical construction sequence consists of drilled shaft installation, construction of the abutment, and placement of fill. Settlement of the fill material along the sides of the abutment, or along the drilled shafts if they are placed through fill, is likely. Evidence is available to show that virtually any fill will settle (compress) to some extent with time under its own weight, particularly if it is not well compacted. Settlement may also occur in the native soil in response to the load of the new fill. In fine-grained soils such as clay, some of the settlement will be time-dependent as a result of consolidation, which can also induce lateral loading on vertical drilled shafts because the soil may squeeze laterally toward the right in the figure.

Examples of soils that will undergo settlement after drilled shaft construction include loose sand, soft to medium stiff clay, recently-placed fill, and soils subjected to earthquake–induced liquefaction. Settlement may occur in a sand stratum of low initial relative density in response to cyclic loading, which can be caused by an earthquake, traffic vibrations, or seasonal fluctuations in the groundwater level. The presence of a surface loading would contribute to the settlement. When a drilled shaft extends through a soft clay layer the tendency for settlement may be minimal if there is no surface loading; however, the addition of fill, such as an approach embankment, or lowering of the groundwater level could induce considerable consolidation settlement that may continue long after the drilled shafts have been installed.

The cases described above are typical of ground conditions leading to downdrag. However, any condition that results in relative downward movement of the ground relative to the drilled shaft has the potential to cause downdrag. Briaud and Tucker (1997) suggest the following criteria for identifying when downdrag may occur. If any one of these criteria is met, downdrag should be considered in the design:

1. Total settlement of the ground surface will exceed 4 inches.
2. Settlement of the ground surface after deep foundations are installed will exceed 0.4 inch.
3. The height of the embankment to be placed on the ground surface exceeds 6 ft.
4. Thickness of the compressible soil layer exceeds 30 ft.

Referring to Tables 8-2 and 8-3 in Chapter 8, drag load (DD) is a component of the Strength I through Strength V load combinations specified by AASHTO (2017a). The design approach implied by AASHTO therefore involves treating drag forces as a load component to be resisted for both geotechnical and structural strength limit state design of deep foundations. However, careful consideration of the downdrag mechanism leads to the conclusion that drag forces are non-existent at the geotechnical strength limit, and should be limited to evaluation of the structural strength limit state only. In this respect, the recommendation given herein differs from the procedure specified by AASHTO. Drag loads develop in response to *relative* downward deformation of the surrounding soil to that of the shaft. Drag loads are not expected to exist at the geotechnical strength limit state, because downward movement of the shaft required to achieve its limiting geotechnical strength would exceed that of the surrounding geomaterials. For a drilled shaft bearing at its tip in a hard or stiff material, thus limiting or preventing downward movement, drag forces can be fully developed, and the factored drag loads should be included with the other axial load components for checking the *structural* strength limit state. It should be noted that, unlike other types of deep foundations such as driven pipe or timber piles, it is unlikely that drag loads would control the structural design of a drilled shaft. A solid reinforced concrete cylinder designed to carry bridge structural loads in compression and/or flexure would typically not require additional strength to accommodate drag loads. Nevertheless, drag loads (DD) do exist in such cases and should be taken into account in combination with the other components of axial load for structural design. Also note that the tip elevation of a drilled shaft

should typically be controlled by the need to penetrate through the strata that pose a downdrag issue, in order to reach a depth beyond the downdrag.

Drag forces due to downdrag (DD) are also included in the Service I Limit State load combinations specified by AASHTO (2017a). Prediction of the downward movement of a drilled shaft is appropriately addressed as part of the geotechnical service limit state design.

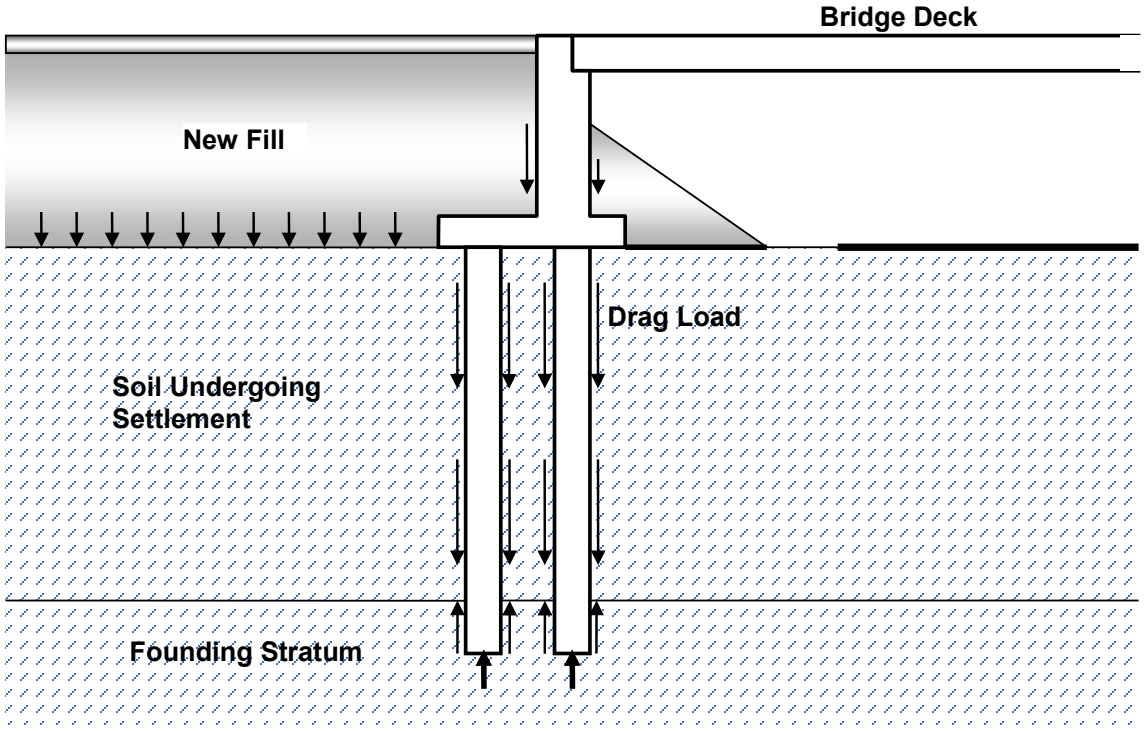


Figure 10-19 Common Sources of Downdrag at Drilled Shaft Supported Bridge Abutments

10.6.2 Downdrag – Basic Concepts and Analysis

A fundamental concept of deep foundation response to downdrag is the "neutral plane," defined as the depth above which the drilled shaft is subjected to drag forces and below which the shaft develops axial resistance to the downward forces (Fellenius, 2006). The downward forces consist of axial compressive load transmitted from the supported structure to the top of the foundation plus drag load. Establishment of the neutral plane is an essential step in any analytical procedure used to evaluate limit states under downdrag. The basic concept is illustrated in Figure 10-20. The left side of the figure shows a free body diagram of a deep foundation under axial compression load (Q) and subjected to settlement-induced drag forces acting downward above the neutral plan, and upward-acting side and base resistances below the neutral plane. The right side of Figure 10-20 shows two load transfer curves: Curve A is constructed by plotting the calculated nominal resistance R_n minus the side resistance as a function of depth. Curve B represents the applied sustained load at the top of the foundation (Q) plus shaft downdrag (assumed equal to side resistance), also plotted as a function of depth. The neutral plane is the depth at which Curves A and B intersect.

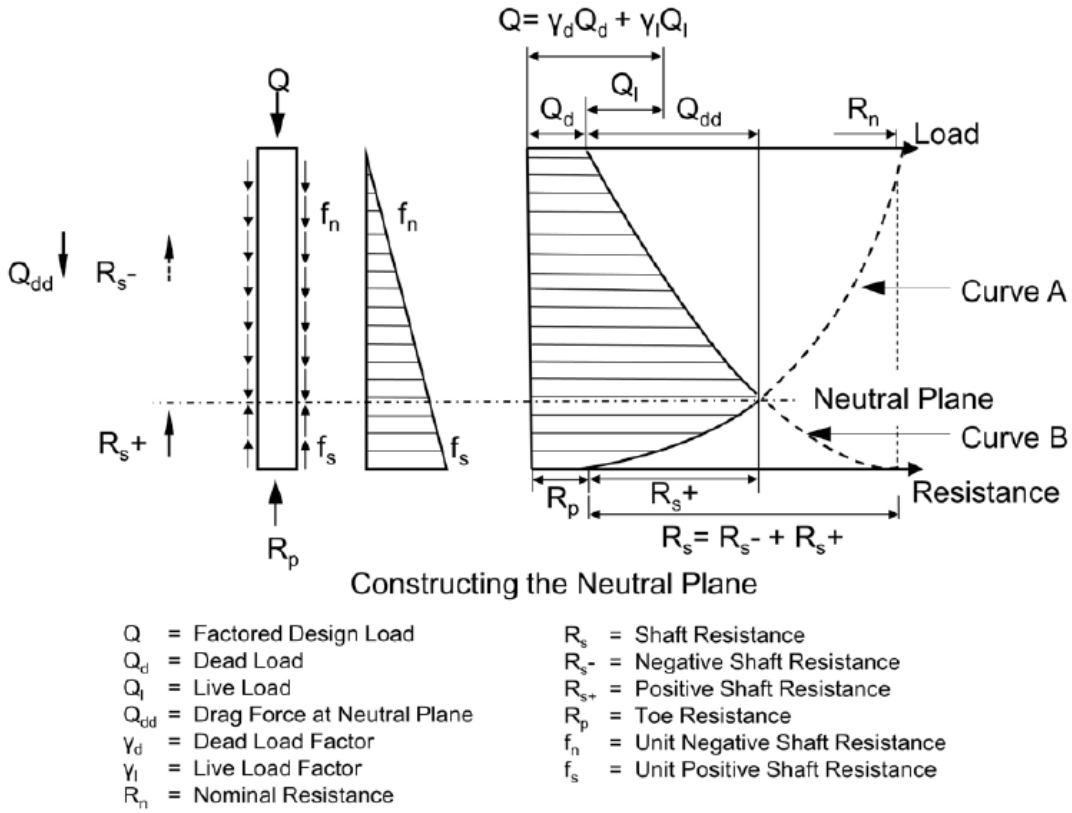


Figure 10-20 Neutral Plane Concept (after Goudrealt and Fellenius 1994)

The analytical approach adopted by the FHWA for design of deep foundations with downdrag is presented in Section 7.3.6.1 of the FHWA publication "GEC 12: Design and Construction of Driven Pile Foundations" (Hannigan et al., 2016) and will not be repeated herein. The recommended approach is based on the neutral plane method proposed by Fellenius (1989) and modified by Siegel et al. (2013). The method is presented within the LRFD framework for evaluating the structural strength and geotechnical service limit states. A step-by-step procedure is presented and requires the following information:

- Nominal (unfactored) sustained axial load demand at top of shaft
- Subsurface stratigraphy with appropriate parameters for each layer
- Soil behavior models to characterize load-deformation response (instrumented load test results, or t-z and q-z models)
- Information on fill placement including amount, lateral extent, and timeline

In the analysis outlined above, the designer must select values of shearing stress imposed by the settling soil layers, referred to herein as nominal unit drag load, f_{DN}. The methods described in Section 10.3.5 for calculating nominal unit side resistances are applicable to the downdrag problem and are recommended for drilled shafts. These include the β-method for cohesionless soils, the α-method for short-term undrained conditions in cohesive soils, and effective stress analysis for long-term conditions in cohesive soils, where appropriate.

Drag loads resulting from post-liquefaction settlement are addressed in Section 10.7, and downdrag effects on drilled shaft groups are considered in Chapter 11.

10.6.3 Strategies to Address Downdrag

To address the potentially negative impacts of downdrag, the axial resistance of a drilled shaft can be increased by increasing its size (depth and diameter), or by adding additional shafts. Note that increasing shaft diameter attracts more drag load, while increasing shaft length adds resistance without increasing drag loads and is more effective in most cases. Alternatively, measures can be taken to reduce downdrag. Some of the measures identified by Briaud and Tucker (1997) for driven piles are also applicable to drilled shafts, including:

- Preloading the soil to induce settlement of the ground prior to drilled shaft construction, thereby reducing settlement that will take place after drilled shaft construction.
- Use of lightweight materials as structural fill in place of conventional fill to reduce surface loading and thereby reduce the magnitude of settlement from compressible soil layers.
- Use of permanent steel casing, with or without surface coating, to reduce soil adhesion within the zone of downdrag.
- Prevent direct contact between the drilled shaft and compressible soils expected to cause downdrag. This approach may be limited to cases in which lateral loading is not significant. Load transfer through side shear can be eliminated or minimized by using permanent double casing in the upper portions of the shaft and filling the annular space between the casings with a material that essentially eliminates shear load transfer, such as styrofoam beads.

Use of ground improvement techniques that increase the stiffness of the compressible soil layers can be another effective means for reducing downdrag.

An increase in drilled shaft diameter, if coupled with a reduction in the number of drilled shafts in a foundation group, could reduce the total downdrag load on the foundation.

For an individual case, both approaches (change the drilled shaft design versus the application of measures to reduce settlement) should be considered; the solution should then be selected on the basis of cost, constructability, and other project requirements.

10.7 SEISMIC CONSIDERATIONS

Methodology for LRFD seismic analysis and design of bridges is provided in the NHI manual by Marsh et al. (2014). The reader is referred to this reference for methods of ground motion analysis, assessment of earthquake hazards including liquefaction, design methods and philosophy, and other aspects of seismic design that result in the load demands relevant to drilled shaft design.

The AASHTO Extreme Event I load combination includes the earthquake (EQ) load component (also see Tables 8-3 and 8-4). The factored axial force effects (axial load demands) transmitted to drilled shaft foundations are provided by the structural design engineer based on modeling of the superstructure under the Extreme Event I load combination. The drilled shafts are then designed to satisfy the LRFD criterion (Equation 10-1), taking into account the factored axial resistances. In general, the expressions for side and base resistances presented in Section 10.3.5 of this chapter are applicable, but with the following seismic issues taken into account:

1. Reductions in effective stress as a result of excess pore water pressure generation
2. Liquefaction or cyclic softening due to seismic excitation

3. Post-liquefaction downdrag
4. Resistance factors accounting for extreme event risk

Effective Stress

For cohesionless soils, unit side resistance is calculated using the beta method (see Equation 10-7) in which the coefficient β is multiplied by the vertical effective stress σ'_v at mid-depth of the layer. In saturated loose granular soils, cyclic shear strain during ground shaking generates excess pore water pressure, reducing effective stress. Because side resistance is proportional to effective stress in the framework of the beta method, a reduction in effective stress due to seismic excitation results in a decrease in side resistance even in the absence of liquefaction. This decreased side resistance should be considered for cohesionless layers for the Extreme Event 1 limit state.

The magnitude of excess pore pressure Δu is often expressed as a fraction of the pre-earthquake vertical effective stress σ'_{v0}, referred to as the excess pore water pressure ratio r_u:

$$r_u = \frac{\Delta u}{\sigma'_{v0}} \qquad 10\text{-}33$$

If the excess pore water pressure approaches the pre-shaking vertical effective stress (*i.e.*, $r_u \rightarrow 1$), effective stress is reduced to essentially zero and liquefaction occurs. However, even in the absence of liquefaction, generation of excess pore water pressure decreases effective stress, reducing side resistance. This can be accounted for by using a modified version of the beta method for the condition $0 \le r_u < 1$ (Boulanger and Brandenberg 2004):

$$f_{SN} = \sigma'_{v0}\left(1 - r_u\right)\beta = \sigma'_{v0}\left(1 - r_u\right)K_o \tan(\delta) \qquad 10\text{-}34$$

Other than r_u, the parameters in Equation 10-34 should be assigned the same values as for pre-earthquake conditions as described in the text following Equation 10-7.

To implement Equation 10-34 it is necessary to estimate r_u using the results of liquefaction triggering analyses (described in Chapter 9). Multiple methods are available for estimating r_u as a function of the factor of safety against liquefaction FS_{liq} for cases of $FS_{liq} > 1$. The simplest approach is a stress-based framework that relates FS_{liq} to r_u directly as presented by Marcuson et al. (1990). The range of data presented by Marcuson et al. (1990) can be approximated by the following hyperbolic best-fit:

$$r_u = \frac{1}{1 + 15\left(FS_{liq} - 1.0\right)} \qquad 10\text{-}35$$

Research has demonstrated that shear strain is more closely associated with pore water pressure generation and liquefaction triggering than shear stress. An alternative approach to Equation 10-35 is to estimate r_u as a function of maximum cyclic shear strain using the following relationship developed by Cetin and Bilge (2012):

$$r_u = \exp\left\{\ln\left[1-\exp\left(-\frac{-0.407\gamma_{\max}}{-0.486+0.025\ln\sigma'_{v0}-\frac{D_R}{100}}\left(\frac{0.620}{1+\gamma_{\max}}+1\right)\right)\right]\right\} \qquad 10\text{-}36$$

where:

D_R = Relative density, in percent,
γ_{max} = Maximum cyclic shear strain

Cetin and Bilge (2012) used results of a parametric laboratory cyclic testing study to demonstrate that the strain-based approach more accurately predicts r_u over a wide range of conditions compared to earlier stress-based approaches. Maximum cyclic shear strain can be computed using the same method described below for downdrag.

This method of estimating reduced side resistance due to excess pore water pressure should be applied for layers with a computed FS_{liq} between 1.0 and 1.5. For $FS_{liq} > 1.5$, significant excess pore water pressure generation is not expected. For critical projects in high seismic regions, the upper limit of FS_{liq} for which pore water pressure generation is considered could be increased beyond 1.5. For layers with r_u close to 1.0, Equation 10-34 predicts very low side resistance. A lower bound side resistance should be taken as equal to the post-liquefaction residual undrained strength of the layer, $s_{u,r}$. Methods for estimating $s_{u,r}$ on the basis of penetration resistance and σ'_{v0} are discussed in Chapter 9.

Strength Reductions

As noted above, if seismically generated excess pore water pressure reaches a threshold value ($r_u \approx 1$), liquefaction is triggered, accompanied by a drastic reduction of shear strength in the affected zones. Axial resistance of liquefied soil is likely some fraction of $s_{u,r}$; however, such low resistance is negligible in comparison to non-liquefied layers and so can be taken as zero for practical purposes when evaluating the Extreme Event I limit state. Rollins et al. (2018) conducted full-scale blast-induced liquefaction testing in soils surrounding various deep foundation types, which confirmed that liquefied soil exhibits essentially zero side resistance. These same tests indicate that non-liquefied soil layers above the zone of liquefaction can exhibit positive side resistance approximately equal to the pre-shaking value. However, AASHTO (2017a) Section 10.8.4 states that side resistance shall be neglected in all layers above the liquefied soil zone, even if these layer(s) do not undergo liquefaction. While the AASHTO approach likely underestimates the actual side resistance, the range of conditions covered by the Rollins et al. (2018) tests is insufficient to make a general prediction of side resistance in non-liquefied soil layers overlying liquefied soil, which would depend on the relative thickness and depths of the liquefied and non-liquefied layer(s) and the severity of liquefaction. Foundation engineers should use judgment before neglecting the side resistance of non-liquefied layers overlying liquefied zones, particularly in the case of a thick stratum of non-liquefied soil overlying a relatively thin layer of soil predicted to undergo liquefaction.

In no case should a drilled shaft be designed with its tip bearing immediately above or in a layer predicted to be susceptible to liquefaction. This requirement could potentially govern the design tip elevation for drilled shafts supporting bridges in seismically active locations.

Cohesive soils under seismically-induced cyclic loading can also undergo strength decreases, with accompanying decreases in side resistance. The process of 'cyclic softening' occurs in saturated cohesive soils in response to cyclic shear strain (Chu et al., 2008). In particular, sensitive clays with low OCR are most susceptible to cyclic softening. From a practical standpoint, the effect of cyclic softening can be approximated by using a reduced value of undrained shear strength, s_u, in conjunction with the alpha method

for calculating side resistance of drilled shafts in cohesive soil layers during seismic loading. A value equal to 80 percent of the CIUC value of s_u (as described in Section 10.3.5.2) provides a reasonable approximation to account for cyclic softening. More explicit procedures for estimating cyclic softening potential are presented in Idriss and Boulanger (2008), ranging from simplified methods using soil index properties to detailed analyses involving cyclic laboratory testing of undisturbed samples, which would not typically be warranted except for major structures in critical seismic zones.

Post-Liquefaction Downdrag

Post-liquefaction reconsolidation settlement of liquefied layers due to excess pore water pressure dissipation as well as settlement of overlying non-liquefied layers results in drag loads on drilled shafts. The magnitude of post-liquefaction reconsolidation settlement can be approximated by procedures given by Ishihara and Yoshimine (1992) and described in Idriss and Boulanger (2008). Maximum drag loads occur at the end of reconsolidation settlement when effective stress has returned to a condition approximately equal to the pre-earthquake effective stress. Drag loads imposed by previously liquefied layers should be computed as 50 percent of the pre-earthquake side resistance calculated using the beta method. This value is based on the results of multiple tests by Rollins et al. (2018) of instrumented deep foundations subjected to blast-induced liquefaction and downdrag. For geomaterial layers above the zone of liquefaction, drag loads should be based on side resistance values corresponding to static conditions.

According to AASHTO (2017a), the resulting drag loads (*DD*) are then included as a component of the permanent loads in Extreme Event Load Combination I. The load factor for liquefaction-induced downdrag forces is 1.0. Drag load associated with settlement by other causes (*e.g.*, consolidation) are not included in extreme event limit state analyses. The AASHTO provision to account for liquefaction-induced downdrag as part of Extreme Event I loading is based on the assumption that post-liquefaction settlement occurs in conjunction with seismic loading of the structure. This provision appears to be misguided, considering that post-liquefaction reconsolidation settlement and the resulting drag loads occur only as excess pore water pressure has dissipated following the earthquake, which can take hours or even days after shaking has stopped. Post-liquefaction drag loads should therefore be considered only for structural strength and service limit states as described in Section 10.6.

Extreme Event I Resistance Factors

In the LRFD criterion (Equation 10-1) a resistance factor of 1.0 is used for compression and a resistance factor of 0.80 is used for uplift of single drilled shafts under earthquake loading. These resistance factors take into account the risk associated with the extreme event (earthquake), which has a return period thought to exceed the design life of the structure, but which has significant consequences. The lower resistance factor for uplift is intended to account for greater uncertainty in uplift resistance, which includes only side and not base resistance.

10.8 SUMMARY

Design for axial loading is one of the major steps in the overall process of design and construction of drilled shafts, as outlined in Chapter 8. In this chapter, the concepts and methods for LRFD design of single drilled shafts under axial loading are presented. For each category of geomaterial, equations are given for calculating nominal values of side and base resistance. Examples are presented to illustrate the application of LRFD limit state checks to the design of drilled shafts under axial compression. An idealized model of load-settlement behavior, based on load test observations, is presented for evaluating the load-settlement behavior of drilled shafts in soil, providing designers with a tool for evaluation of the Service I limit state. More rigorous methods for analyzing load-displacement behavior are covered in Appendix C.

An approach for evaluating scour caused by the scour design flood and the scour design check (Extreme Event) flood and its effects on foundation resistances is summarized. Basic principles are also presented for design of drilled shafts under uplift, downdrag, and Extreme Event I (earthquake) loading.

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CHAPTER 11
DESIGN OF GROUPS OF DRILLED SHAFTS

11.1 INTRODUCTION

Although one of the advantages of drilled shaft foundations is the small footprint afforded by the use of a single shaft foundation under a single column, there are many occasions in which groups of drilled shafts as shown in Figure 11-1 might be used. The design of drilled shaft groups is not substantially different from the design of other types of deep foundations, except that the use of battered shafts is relatively rare. Also, there are some considerations related to the unique aspects of construction of drilled shafts, and drilled shafts tend to be larger in diameter and stronger in flexure than groups of other types of deep foundations. Drilled shafts can sometimes be used in closer spacing if required by specific constraints of a project site.



Figure 11-1 Group of Drilled Shafts During Construction of the Benetia-Martinez Bridge near San Francisco

This chapter outlines methods for designing groups of drilled shafts based on the techniques described in the previous chapters for designing individual shafts. The focus of this chapter is on the specific considerations used to account for group effects in axial and lateral loading, and the computation of loads to individual shafts due to combined axial, shear, and overturning moments applied to groups of shafts.

11.2 GROUP VERSUS SINGLE SHAFT FOUNDATION

The use of groups of drilled shafts should be considered in cases where foundation loads make single shaft foundations unusually large and particularly costly. As illustrated in Figure 11-2, large overturning moments are most effectively resisted using groups of shafts because of the efficient moment resistance produced by the axial resistance of widely spaced shafts within a group. Because of the need for large, heavy equipment to install single shaft foundations larger than 8 ft in diameter, groups of smaller diameter shafts may be more cost effective in many circumstances.

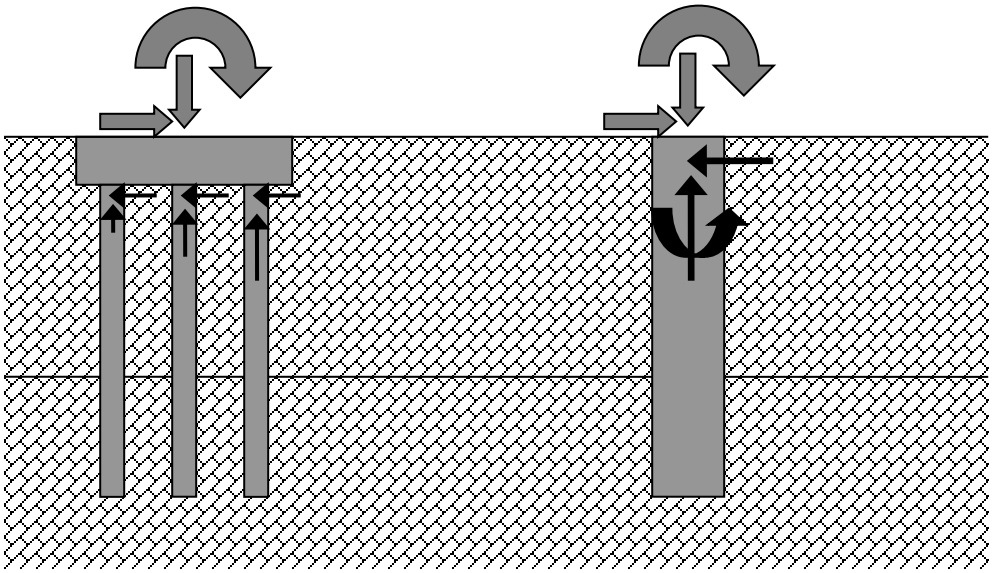


Figure 11-2 Group Versus Single Shaft

Group foundations are less advantageous where the cost or difficulty associated with construction of a pilecap may be significant. In areas where the foundation footprint must be limited to fit in or around existing structures, group foundations can present access problems. For over-water foundations, the construction of a cap can be a major expense and more time-consuming to construct. In addition, scour may be less severe with a smaller footprint of a single shaft compared to a group.

In individual cases where the relative advantages of a group or single shaft foundation are not obvious, the designer should prepare a preliminary or conceptual design of both alternatives for evaluation of costs, risks, schedule considerations, and other factors. Local chapters of contractor trade associations are typically available for consultation on constructability and cost issues. Many state DOT agencies have regular meetings with such groups for consultation on these issues.

11.3 CONSIDERATIONS FOR SPACING

A minimum spacing of 3 diameters center to center between shafts (2 diameters clear space) is typical of routine practice within the industry. There are occasions where 2.5 diameters on center (1.5 diameters clear) can be advantageous, although the efficiency of the group against overturning moment is reduced as the shaft spacing is reduced. At close spacings, the sequencing of construction operations must be planned to avoid the potential for communication between shafts during excavation and concrete placement. In addition, the drilling of a hole less than 3 diameters on center from an adjacent existing

shaft can result in a reduction in lateral stress and/or loosening of the ground around that shaft in some types of materials. Advancement of a casing ahead of the shaft excavation is one means to mitigate potential adverse effects.

In any case, group effects must be considered at a center to center spacing of less than 4 diameters for axial resistance and less than 5 diameters for lateral resistance.

11.4 GROUP EFFECTS ON AXIAL RESISTANCE

The resistance of a drilled shaft group to vertical load is not necessarily the sum of the axial resistance of the individual shafts within the group. In shaft (or pile) groups, the zone of influence from an individual drilled shaft may intersect with other shafts, depending on the shaft spacing, as illustrated in Figure 11-3. Evaluations of shaft group strength (geotechnical strength limit state) should also consider potential block failure of the group, and the potential contribution of the cap to bearing capacity of the shaft group system (termed occasionally as a piled raft). The designer should be aware that settlement of a shaft group may often exceed that which would be predicted based upon a single drilled shaft analysis.

Besides the effect of overlapping zones of influence, effects of construction on ground conditions in and around the group can be significant. Excavated deep foundation elements (such as drilled shafts and continuous flight auger piles), generally tend to decrease the effective stress of the surrounding soil, or at best maintain it at the at-rest (K_o) condition. Changes in effective stress are more pronounced in cohesionless soils. Poorly controlled shaft construction can result in soil loosening during drilling and adversely reduce the lateral stress around previously installed shafts. Some techniques used for drilled shaft construction, such as casing driven in advance of the excavation, may result in densification rather than loosening. In comparison, some types of deep foundations such as driven displacement piles may tend to increase the relative density and effective stress of the surrounding soil.

Effects of construction on ground conditions are less significant in stable geologic formations such as rock, cemented sands, stiff clays, and strong cohesive geomaterials such as marl or shale. Although these materials tend to stand in an open hole with less tendency to loosening during the short period of excavation for a drilled shaft, it is important to note the macro-structure of the formation including the potential for geologic discontinuities which can result in communication between shafts during construction. Uncemented sand layers, seams of weathered or decomposed rock, water-bearing fracture zones, and solution cavities can lead to difficulties during installation of closely spaced shafts. These features require careful control of construction operations and sequencing in order to avoid unpredictable adverse consequences to drilled shaft performance.

11.4.1 Group Effects on Strength (Geotechnical Strength Limit State)

Group effects on the geotechnical strength limit state should be considered for load cases in which the drilled shafts within the group are loaded simultaneously to a large factored axial force effect. The maximum factored load for design of individual drilled shafts in the group is likely to be controlled by load cases with significant overturning moments whereby a corner shaft is subject to a much higher force than neighboring drilled shafts. Often, a quick check of group effects for the load case with maximum overall group load will verify that the design of the individual drilled shaft is controlled by overturning as described in Section 11.6 of this chapter.

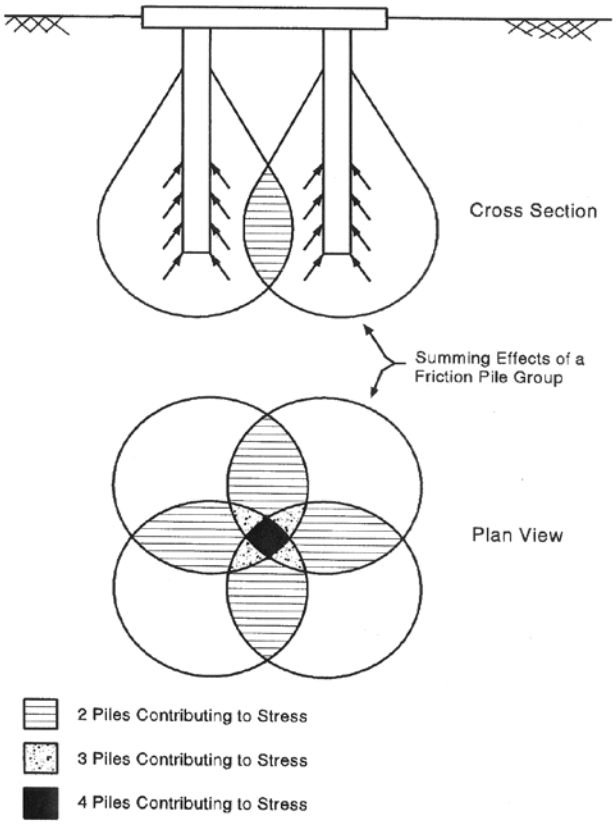


Figure 11-3 Overlapping Zones of Influence in a Frictional Pile Group (after Bowles, 1988)

The efficiency of a pile group (η_g) is often defined as:

$$\eta_g = \frac{R_{ng}}{\sum_{i=1}^{n} R_{n,i}} \qquad 11\text{-}1$$

where R_{ng} is the nominal resistance of the shaft group, and $R_{n,i}$ is the nominal resistance of a single drilled shaft "*i*" in the shaft group with a total of n shafts in the group.

The overlapping zones of influence from individual drilled shafts in a group, and the tendency for the pile cap to bear on the underlying soils (if in contact) tend to cause the drilled shafts and pile cap system and the soil surrounding the drilled shafts to act as a single unit and exhibit a block type failure mode. The group capacity should be checked to see if a block-type failure mode controls the group capacity, as will be discussed subsequently.

Block failure mode for drilled shaft groups generally will only control the design for drilled shaft groups in soft cohesive soils or cohesionless soils underlain by a weak cohesive layer. Note that closer spacing of the drilled shafts in the group will also tend to increase the potential for the block failure mode.

11.4.1.1 Cohesive Geomaterials

For cohesive geomaterials in which the installation of the foundations is not considered to have a significant effect on the in-situ soil and state of stress, the resistance for the geotechnical strength limit state should be determined from the lesser of a block failure mode or the sum of the individual shaft resistances. That is, the efficiency cannot exceed 1.0 as shown in Equation 11-2. The nominal resistance of the block (R_{Block}) is estimated as described below, while the individual drilled shaft nominal resistance ($R_{n,i}$) is estimated as described in Chapter 10.

$$\eta_g = \frac{R_{Block}}{\sum_{i=1}^{n} R_{n,i}} \leq 1 \qquad 11\text{-}2$$

The resistance of the block failure (R_{Block}) mode can be simply estimated as the sum of the side shear contribution from the peripheral area of the block, as shown in Figure 11-4, and the bearing capacity contribution from the block footprint area:

$$R_{Block} = f_{\max} \cdot \left[2 \cdot D \cdot (Z + B)\right] + q_{\max} \cdot (Z \cdot B) \qquad 11\text{-}3$$

where: D, Z, and B are the depth, length, and width of the block, respectively. The nominal unit side resistance of the block, f_{max}, is conservatively computed as if the peripheral surface of the block is a drilled shaft and the base resistance, q_{max}, is computed using the appropriate procedure for cohesive materials as outlined in Chapter 10. However, the nominal unit base resistance of the block must take into account that the zone of influence for the block is deeper than for a single drilled shaft. This effect may be included by assuming a zone of influence below the block to a depth of approximately 2 to 3 times the smaller of block length *Z or width B*, and determining q_{max} by the conventional bearing capacity methods presented in Chapter 10 for this deeper zone of influence.

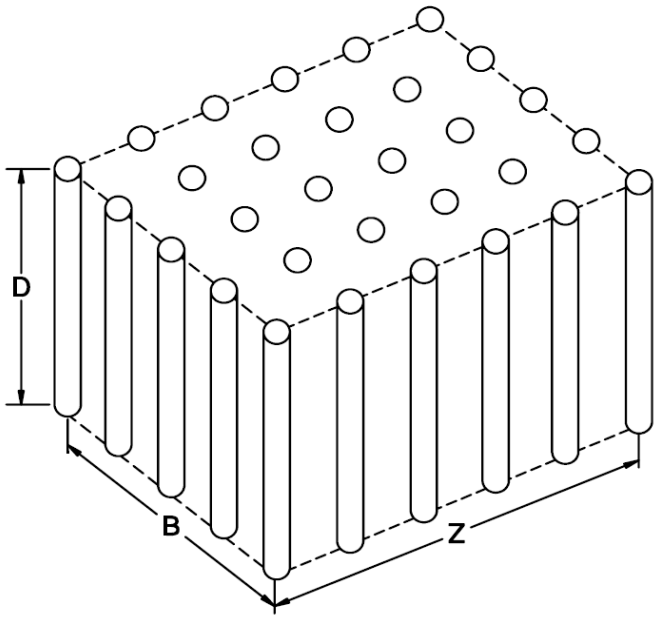


Figure 11-4 Block Type Failure Mode (after Tomlinson, 1994)

11.4.1.2 Cohesionless Soils

AASHTO (2017a) provisions for group efficiencies for drilled shafts in cohesionless soils (AASHTO 10.8.3.6.3) states that for cases in which the cap is not in contact with the ground:

η = 0.65 for a center-to-center spacing of 2.5 diameters

η = 0.80 for a center-to-center spacing of 3 diameters

η = 1.0 for a center-to-center spacing of 4 diameters or more

The value of η may be determined by linear interpolation for intermediate spacings. If the cap will be maintained in contact with the ground (which is not typically applicable to bridge foundations), then no reduction is applied. Pressure grouting along the sides and base is also cited as a measure which mitigates the need for reduction for group effects in cohesionless soil. Additionally, the commentary provided in AASHTO Section 10.8.3.6.1 states that if casing is advanced in front of the drilled shaft excavation, this reduction need not be made.

As noted in Section 11.4.1, any such group reduction factor would apply to the overall (or average) axial resistance of the drilled shafts in the group subject to the total (or per shaft average) load on the group, and not to the maximum load on any shaft resulting from an overturning force.

The reductions for cohesionless soil are recommended in large part on the basis that the installation of drilled shafts in cohesionless soils may result in loosening of the soil and/or stress relaxation during installation. The provision cited in the AASHTO commentary of 10.8.3.6.1 as referenced above appears to suggest that casing advanced in front of the excavation heading mitigates this concern; however, the discussion of construction techniques provided in Chapter 5 illustrate how stability can be maintained with a variety of techniques and likewise loosening due to instability can also occur even if casing is advanced ahead of the excavation. The key to avoiding loosening of the ground is that stability be maintained using appropriate construction methods; lack of stability during construction can potentially create ground disturbance that causes greater reduction in axial resistance than indicated by the values cited above.

There is evidence that the AASHTO recommended group efficiency values are most likely conservative when the cohesionless soil is not loosened by the installation process. Where the means and methods of drilled shaft installation are not defined during design and the quality of the work uncertain, the use of the AASHTO group reduction values cited in Section 10.8.3.6.3 represent a prudent approach for design.

Results from small-scale field tests on small bored piles in cohesionless soils from diverse locations around the world suggest that an efficiency of 1.0 or greater may be obtained with pile center-to-center spacing of approximately 3 to 4 diameters. Note that a typical center-to-center spacing of 3 pile diameters would result in a recommended efficiency of 0.80 using the AASHTO provisions cited above.

Studies of groups of small bored piles in cohesionless soils include Garg (1979), Liu et al. (1985), Senna et al. (1993) and Ismael (2001). The piles did not exceed the range in diameter of 5 to 13 inches, and in length from 8 to 24 times their respective diameter in all four cited studies. Note that all four of the studies were performed in either dry sand or sand with fines above the water table. Efficiencies for groups in clean sands below the water table may be lower than reported in the cited studies due to a greater potential for relaxation of lateral stress.

Results of finite element studies of groups of bored piles in an elastoplastic material with a Drucker-Prager yield surface to simulate a c-φ soil were reported by Katzenbach and Moormann (1997). These studies provide a model of the effect of stress overlap absent any installation effects on the soil strength

properties or state of stress. The FE model results suggest that piles at a spacing of 3D on center are capable to provide resistance in excess of the maximum resistance mobilized by a single standing pile, albeit at relatively large displacement (Figure 11-5). The pile resistance shown on Figure 11-5 for model 1 is at 3D center-to-center spacing and at 6D spacing for model 2. Note that both models of groups mobilize less resistance at small vertical displacement, and thus serviceability (settlement) is affected differently than the geotechnical strength limit state.

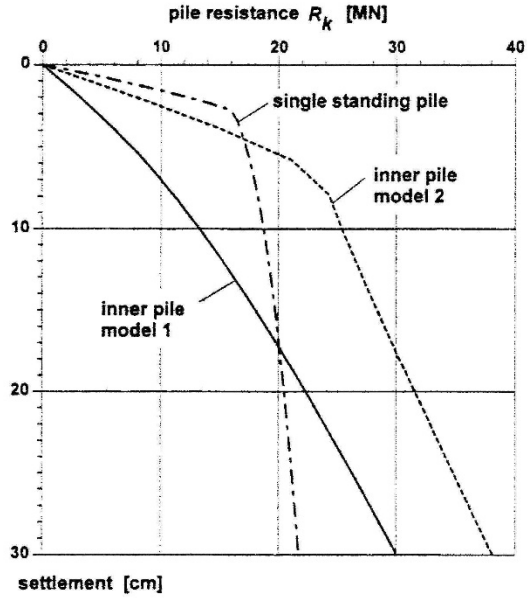


Figure 11-5 Axial Resistance from FE Model Results of Bored Pile Groups (Katzenbach and Moormann, 1997). 1 MN ≈ 225 kips; 2.54 cm ≈ 1 inch.

While these studies have limitations with respect to drilled shaft design in cohesionless soils, they suggest that there may be circumstances in which the AASHTO specifications would result in a conservative estimate of group resistance at the geotechnical strength limit state. The potential adverse group effects of drilled shaft installation in cohesionless soil are more likely to occur if there are reductions in lateral stress and/or reductions in soil relative density.

Effects of drilled shaft installation on soil density or stress could be reflected in post-construction in-situ tests (SPT or CPT) within the shaft group. Likewise, verification tests of an interior shaft should provide a representative indication of a typical shaft within a group after installation of the entire group. If reliable interpretations from a well conceived testing program can verify that negative group effects are less severe than indicated by the AASHTO recommendations for drilled shafts, then an alternate approach may be justified on a project-specific basis.

11.4.1.3 Drilled Shaft Group in a Strong Layer with a Weak Underlying Layer

If a weak formation is present below the foundation layer, the group efficiency should be checked to ascertain whether a group efficiency of less than 1 is warranted. The group efficiency may be checked as described in Section 11.4.1.1, where the individual drilled shaft nominal resistance ($R_{n,i}$) is estimated as described in Chapter 10 and the block is assumed to extend to the weak layer. It should be noted that a weak layer below the shaft group will, in most cases, present a significant consideration from the

standpoint of group settlement as outlined in Section 11.6. Settlement considerations may require that minimum drilled shaft penetration be achieved to an elevation below the compressible layer.

11.4.2 Settlement of Shaft Groups (Serviceability Limit)

The development of resistance with displacement of individual shafts was discussed in general in Chapter 10. Where groups of drilled shafts are subject to significant vertical loading, the settlement of a shaft group is likely to be significantly greater than the settlement of an individual shaft at the same average load, especially for cases where the soils below the shaft bearing layer are compressible.

The greater settlement of the shaft group is attributed to a deeper zone of influence for the group than that for a single shaft, as illustrated in Figure 11-6.

Settlement of shaft groups can be attributed to a combination of elastic compression of the shafts, and settlement of the surrounding soils. Settlement of the surrounding soils will primarily consist of nearly instantaneous compression for purely cohesionless soils, and primarily time dependent consolidation for purely cohesive soils. Note that layered systems of soils may contain appreciable amounts of both immediate compression and consolidation settlements.

It is worth noting that designers who must consider the effects of pile foundation settlements should carefully consider the magnitude and timing of the application of those loads and their effect on the structure. For instance, the dead load of the column, pier cap, and perhaps other portions of the bridge structure may be in place and settlement due to these loads may be complete before the final connections of any settlement-sensitive portions of the structure are made. It may be that only settlement from the additional loads imposed after the girder bearing plates are set are the movements with significant consequences to the structure.

Simplified methods for estimating pile group settlement are presented in the following sections. The methods presented were formulated for use with driven pile groups and are considered to be generally applicable to drilled shaft group settlements. The deeper zone of influence for a deep foundation group is unlikely to be significantly affected by the type of the deep foundation elements, although differences in individual pile or drilled shaft stiffness and mobilization of capacity can affect settlements to some degree.

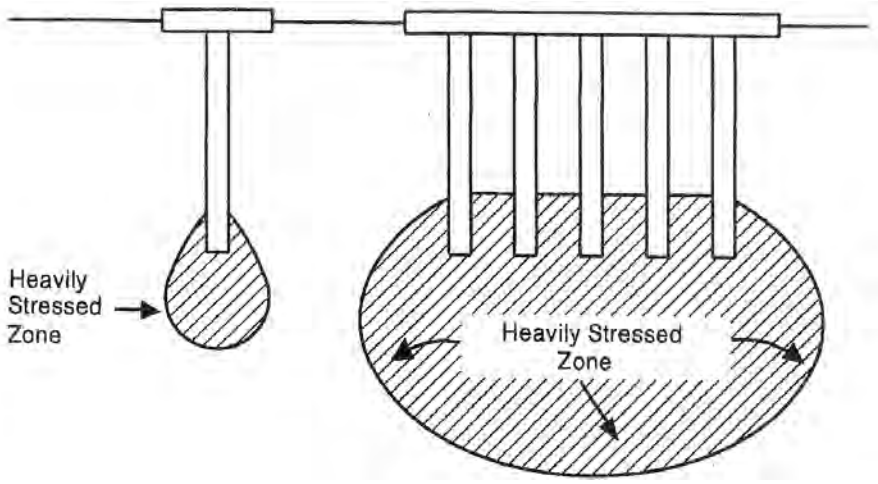


Figure 11-6 Deeper Zone of Influence for End Bearing Shaft Group (after Tomlinson, 1994)

11.4.2.1 Elastic Compression of the Shaft

Elastic compression of the drilled shaft is a function of the imposed load, the shaft stiffness, and the load transfer characteristics from the shaft to the surrounding soil. For many practical problems, a drilled shaft may be considered "rigid" if its stiffness ratio (*SR*) as defined in Equation 11-4, is less than approximately 0.010. In such cases, the elastic shortening of the shaft is likely to be very small compared to the settlement related to the soil in which the drilled shaft is embedded. Otherwise, elastic compression should be estimated and included in settlement calculations, as well as subtracted from shaft displacement when determining the mobilization of either side or base resistance at values less than the strength limit.

$$SR = \left(\frac{L}{B}\right) \cdot \left(\frac{E_{soil}}{E_{pile}}\right) \qquad \leq 0.010 \qquad \text{11-4}$$

where:
L = shaft embedment depth
B = shaft diameter
E_{soil} = average Young's modulus of the soil
E_{pile} = Young's modulus of the drilled shaft

The elastic compression of a shaft (Δ) may be calculated as the sum of elastic compression of "n" shaft segments as follows:

$$\Delta = \sum_{i=1}^{n} \frac{Q_i \cdot L_i}{A_i \cdot E_i} \qquad \text{11-5}$$

L_i, A_i, and E_i are the length, average cross-sectional area, and average composite modulus, respectively, for each of the shaft segments. Q_i is the average axial load at the shaft segment. The load at the top shaft segment would be the total imposed load to that individual shaft, and would reduce in magnitude down to the mobilized end bearing load at the shaft tip in accordance with the load transfer response of the shaft to soil system. If downdrag or uplift are present, the load distribution would be as described in Chapter 10.

The load imposed to the individual drilled shaft could become a complex solution if the pile cap were to provide a contribution to the total capacity of the shaft group system (i.e. a piled raft), and the group was subject to eccentric loading effects. However, to estimate the load imposed to the individual shaft for purposes of elastic compression calculations, it may be sufficient to simply divide the total load of the shaft group by the number of shafts.

For many practical problems, an estimate of elastic shortening may be made using simplified assumptions regarding the load distribution in the pile. For example, a constant load transfer rate (i.e. a uniform unit side shear along the entire length of the pile) and axial load supported entirely in side friction would result in a triangular distribution of load in the shaft versus depth ranging from the maximum load at the shaft top to 0 load at the shaft toe. For this condition, the elastic compression may be computed as:

$$\Delta = \left(\frac{1}{2}\right) \cdot \frac{Q_{\max} \cdot L_{shaft}}{A_{shaft} \cdot E_{shaft}} \qquad \text{11-6}$$

An upper bound (other than the possibility of downdrag) is represented by a shaft acting as a free standing column with no load transfer along the entire length of the shaft and the total maximum imposed load to the shaft supported in end bearing. For this condition, Equation 11-7 provides an upper bound estimate of elastic shortening in the pile. Note that downdrag or soil swell conditions could present a more significant pile load, and for such a case Q_{max} would be determined as described in Chapter 10.

$$\Delta_{\max} = \frac{Q_{\max} \cdot L_{shaft}}{A_{shaft} \cdot E_{shaft}} \qquad 11\text{-}7$$

Equations 11-6 and 11-7 can be used to quickly estimate the potential magnitude of elastic shortening, and determine if more complete evaluation of load distribution is justified for the purpose of computing settlement.

11.4.2.2 Compression Settlement in Cohesionless Soils

Meyerhof (1976) recommended that the compression settlement of a pile group (S_{group}) in a homogeneous sand deposit (not underlain by a more compressible soil at greater depth) be conservatively estimated by the correlations to either SPT *N*-values (blows/ft) or to CPT q_c (tip bearing). Note that if the group were to be underlain by cohesive deposits, time dependent consolidation settlements would be needed as described in the following section. The method proposed by Meyerhof (1976) does not distinguish 60 percent hammer efficiency for the N-values. However, the 60 percent correction is recommended.

For SPT *N*-values in cohesionless soils:

for sands:

$$S_{group} = \frac{4 \cdot p_f \cdot I_f \cdot \sqrt{B}}{\overline{N_{60}}'} \qquad 11\text{-}8$$

for silty sands:

$$S_{group} = \frac{8 \cdot p_f \cdot I_f \cdot \sqrt{B}}{\overline{N_{60}}'} \qquad 11\text{-}9$$

For CPT q_c values in cohesionless saturated soils:

$$S_{group} = \frac{p_f \cdot I_f \cdot B}{2 \cdot \overline{q_c}} \qquad 11\text{-}10$$

where:

S_{group}	=	estimated total settlement (inches)
p_f	=	foundation pressure (ksf), group load divided by group area (plan view)
B	=	width of drilled shaft group (ft.)
D	=	drilled shaft embedment depth below grade (ft)
I_f	=	influence factor for group embedment = $1 - [\, D/(8B)\,] \geq 0.5$

$\overline{N'_{60}}$ = average corrected SPT N-value (bpf) within a depth B below the shaft tip

$\overline{q_c}$ = average static cone tip resistance (ksf) within a depth B below the shaft tip

11.4.2.3 Consolidation Settlement in Cohesive Soils

Consolidation settlement of cohesive soils is generally associated with sustained loads and occurs as excess pore pressure dissipates (primary consolidation). For purposes of discussion in this section, the time rate of settlement will not be addressed directly. Design for a total magnitude of settlement for the full sustained dead load on the structure would represent a conservative approach to settlement in cohesive soils. For most structures, a portion of the dead load will be in place (pile cap, column, pier cap, etc.), and consolidation for that portion of the load may be nearly complete before settlement-sensitive portions of the structure (above the girder bearing plates) are in place. Should computed settlement for total sustained dead load be found to significantly affect the design, it may be prudent to evaluate the time rate of the settlement for construction loads to more accurately assess the post-construction settlements. Time rate of primary consolidation is a topic covered in most geotechnical texts and in FHWA training materials (FHWA report No. HI-88-009 by Cheney and Chassie, 2002).

The consolidation settlement is driven by the load exerted on the shaft group and resulting stress distribution in the soil below and around the shaft group. The actual stress distribution in the subsurface can be affected by many factors including the soil stratigraphy, relative shaft/soil stiffness, shaft to soil load transfer distribution, pile cap rigidity, and the amount of load sharing between the cap and the drilled shafts. For most practical problems, a simplified model of stress distribution is sufficient to estimate shaft group settlement. The equivalent footing method is presented below as a simplified method to estimate vertical stress with depth in the soil below the drilled shaft group.

Terzaghi and Peck (1967) proposed that pile group settlement could be estimated using an equivalent footing situated 1/3 of the pile embedment depth (D) above the pile toe elevation, and this equivalent footing would have a plan area of the pile group width (B) by the pile group length (Z). The pile group load over this plan area is then the bearing pressure transferred to the soil through the equivalent footing.

The same load is then assumed to spread within the frustum of the pyramid of side slopes of 1(horizontal): 2(vertical), thus reducing the bearing pressure with depth as the area increases (P_d as a function of depth). This concept is illustrated in Figure 11-7.

In some cases, the depth of the equivalent footing should be adjusted based upon soil stratigraphy and load transfer mechanism to the soil, rather than fixing the equivalent footing at a height of 1/3 D above the shaft toe for all soil conditions. Figure 11-8 presents the recommended location of the equivalent footing for a variety of load transfer and soil resistance conditions.

The cohesive soils below the equivalent footing elevation are broken into layers, and the total consolidation settlement is the sum of the consolidation settlement of all the layers. Note that multiple laboratory curves may need to be generated to accommodate the different layers depending on the soil consistency and maximum past pressures. The settlement of each layer may be calculated as presented in Equations 11-11 through 11-13. A generic example of this consolidation curve is shown in Figure 11-9 to illustrate the terms in these equations.

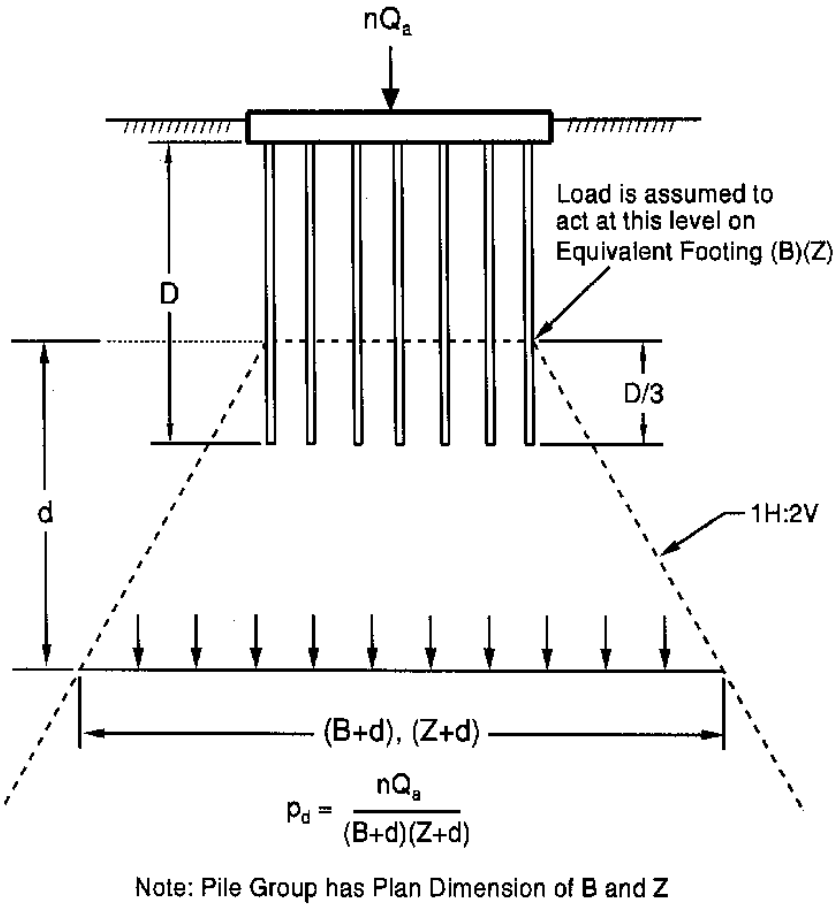
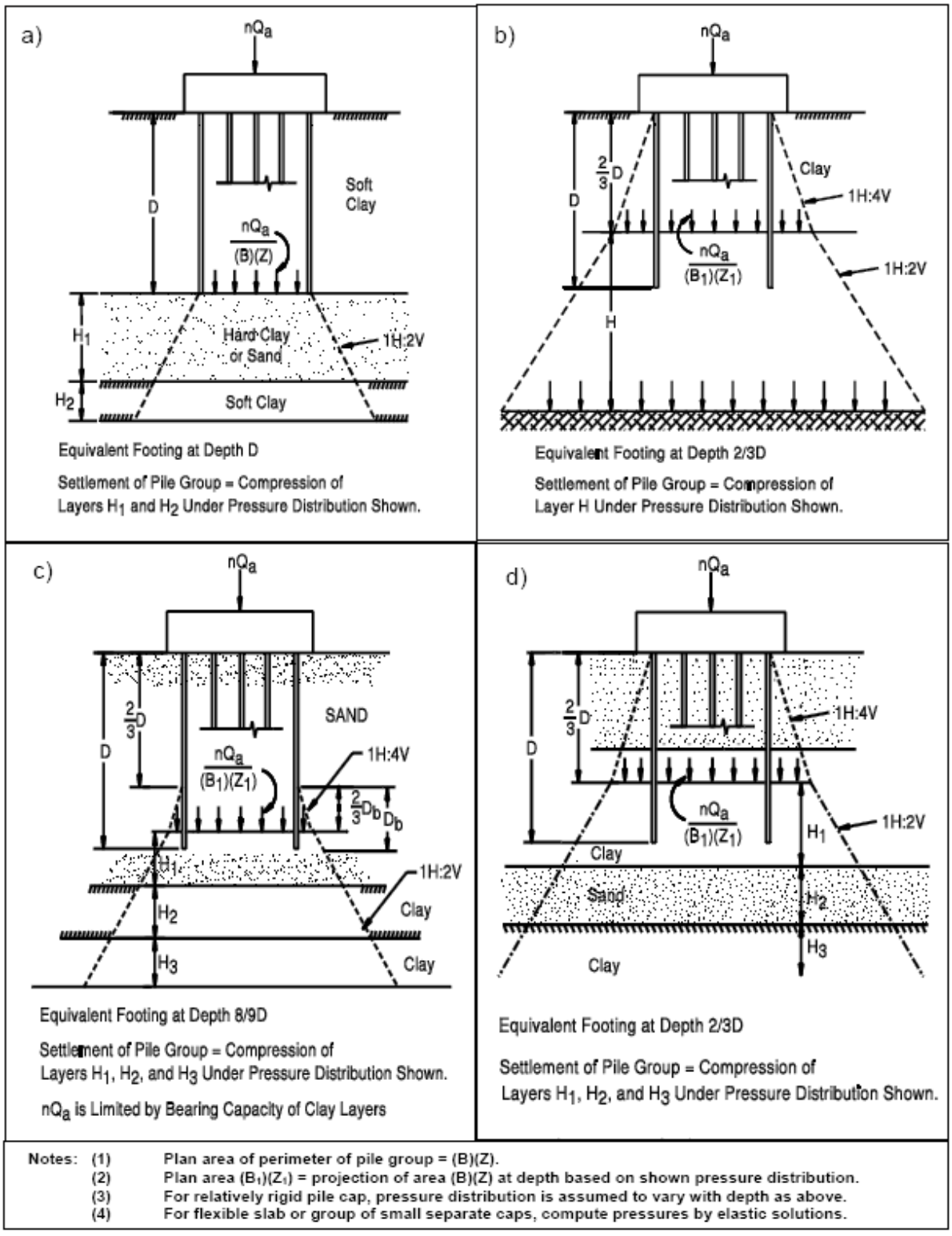


Figure 11-7 Equivalent Footing Concept for Pile Groups (after Terzaghi and Peck, 1967).



Notes:
(1) Plan area of perimeter of pile group = (B)(Z).
(2) Plan area $(B_1)(Z_1)$ = projection of area (B)(Z) at depth based on shown pressure distribution.
(3) For relatively rigid pile cap, pressure distribution is assumed to vary with depth as above.
(4) For flexible slab or group of small separate caps, compute pressures by elastic solutions.

Figure 11-8 Pressure Distribution Below Equivalent Footing for Pile Group (adapted from Cheney and Chassie, 2002).

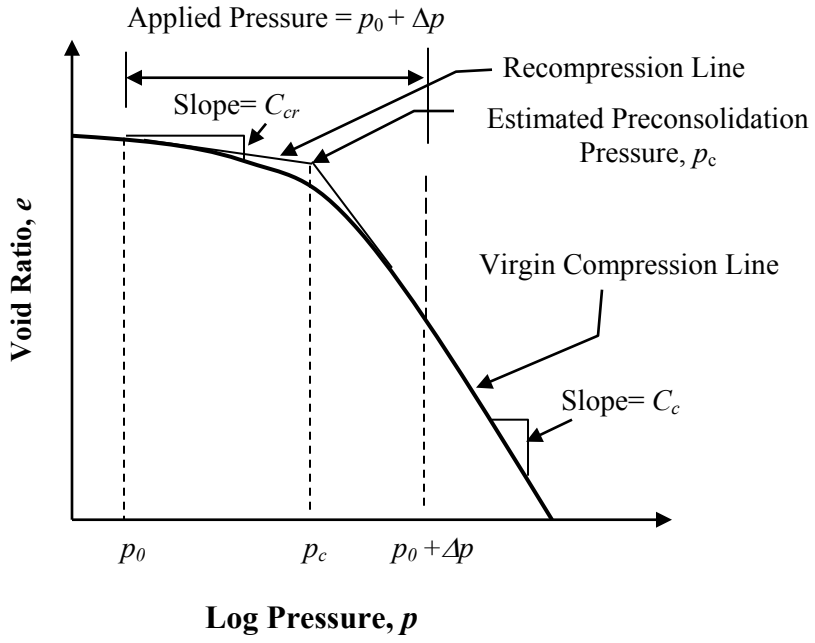


Figure 11-9 Typical *e* versus Log *p* Curve from Laboratory Consolidation Testing.

The settlement for an overconsolidated cohesive soil layer (S_i) where the pressure after application of the foundation load is greater than the soil preconsolidation pressure ($p_o + \Delta > p_c$):

$$S_i = H \cdot \left[\frac{C_{cr}}{1+e_0} \log\left(\frac{p_c}{p_0} \right) \right] + H \cdot \left[\frac{C_c}{1+e_0} \log\left(\frac{p_0 + \Delta p}{p_c} \right) \right] \qquad 11\text{-}11$$

The settlement for an overconsolidated cohesive soil layer (S_i) where the pressure after the foundation pressure increase is less than the soil preconsolidation pressure ($p_o + \Delta < p_c$):

$$S_i = H \cdot \left[\frac{C_{cr}}{1+e_0} \log\left(\frac{p_0 + \Delta p}{p_0} \right) \right] \qquad 11\text{-}12$$

The settlement for a normally consolidated cohesive soil layer ($p_o = p_c$):

$$S_i = H \cdot \left[\frac{C_c}{1+e_0} \log\left(\frac{p_0 + \Delta p}{p_0} \right) \right] \qquad 11\text{-}13$$

where:

S_i = total settlement for layer
H = original thickness of stratum or layer
C_c = Compression index
C_r = Recompression index

e_0 = Initial void ratio
p_o = effective overburden pressure at midpoint of stratum prior to pressure increase;
p_c = estimated preconsolidation pressure
Δp = average change in pressure

Note that if the soil were underconsolidated ($p_o > p_c$), as from the placement of substantial fill on the site, the consolidation process due to loads imposed prior to placement of the foundation would still be continuing. This condition would result in an additional downdrag load to the shaft group as discussed in Chapter 10.

11.4.3 Group Effects in Rock and Cemented Soils

Because of the great axial resistance provided by individual drilled shaft foundations in rock, cemented materials and strong cohesive soils, large groups of drilled shafts are not often required for typical transportation structures founded in these materials. Where groups of drilled shafts are employed, the methods for evaluation of geotechnical strength described for cohesive soils may be used for design; however, because the strength of the rock (or strong material) is usually greater than the strength at the shaft/rock interface, group effects would rarely be expected to control design. Superposition of stresses from adjacent drilled shafts may result in increased deformations of groups of shafts relative to that of isolated individual drilled shafts subject to similar loads; however, settlement of drilled shafts founded on rock and strong cohesive soils are usually very small and group effects are not usually significant for transportation structures.

11.5 GROUP EFFECTS IN LATERAL LOADING

When laterally loaded drilled shafts are used in closely-spaced groups, a given shaft will deflect further under a given set of loads than if loaded when the neighboring shafts are not present and bending stresses will increase beyond those that occur when neighboring shafts are not present. It is therefore important to consider group effects due to loading when shaft spacing is less than about six diameters in any direction.

11.5.1 P-multiplier Concept

Brown et al. (1987) showed that the behavior of a pile within a 3 X 3 group of free-headed laterally loaded piles with a 3-diameter spacing could be modeled with the same software that is used to analyze a single laterally loaded pile or drilled shaft, provided the p-y curves were scaled with a "p- multiplier," $\boldsymbol{P_m}$, as illustrated in Figure 11-10. In this approach, all of the values of soil resistance p are multiplied by a factor that is less than 1 (the multiplier), where the factor applied to any individual drilled shaft depends upon the location of the shaft within the group and the spacing of the shafts within the group. That is, all along the p-y curve:

$$\boldsymbol{p}_{group\ shaft} = \boldsymbol{P_m}\, \boldsymbol{p}_{single\ shaft} \qquad 11\text{-}14$$

This factor reflects a dominant physical situation that develops within a laterally loaded group of drilled shafts or piles: the drilled shafts in the leading row push into the soil in front of the group. The soil reacting against any drilled shaft in this "front row" is relatively unaffected by the presence of the other

drilled shafts in the group and only a minor adjustment needs to be made to the p-y curves. However, the shafts in the rows that "trail" the front row are obtaining resistance from soil that is being pushed by the shafts into the voids left by the forward movement of the drilled shafts in front of them. This phenomenon causes the value of soil resistance, p, on a p-y curve to be reduced at any given value of lateral deflection, y, relative to the value that would exist if the drilled shafts in the forward row were not there. In addition, the presence of all of the shafts in the group produces a mass movement of the soil surrounding the shafts in the group, which reduces the p-value for a given displacement, y, to varying degrees for all drilled shafts in the group.

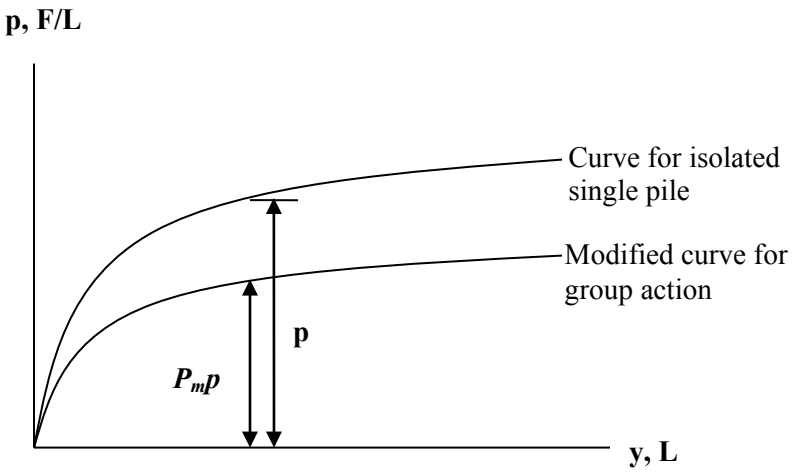


Figure 11-10 The P-Multiplier Concept (Brown et al., 1987)

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Subsequent to these initial field tests, there have been a number of field loading tests, centrifuge model tests, and finite element modeling of simulated tests, the results of which validate the general concept and understanding that the behavior is dominated by row position (Brown et al., 1988; Brown and Shie, 1991; McVay et al,, 1995; Pinto et al., 1997; Ruesta et al,, 1997; Rollins et al,, 1998; Ashford and Rollins, 1999; Brown et al,, 2001). A range of values of P_m have been observed and reported, which suggest the following trends:

1. The greatest reduction in soil resistance occurs between the first row and trailing rows; differences between trailing rows are generally small.
2. Variations in P_m related to soil type are relatively minor; there is some evidence that the effect of row position is somewhat more pronounced in sands than in clays, but differences related to soil type may be less significant than differences related to installation effects and typical spatial variability of soil properties.
3. Installation of driven displacement piling may influence the distribution of soil resistance within a group in sands because of densification (i.e., higher P_m in trailing rows than would otherwise be expected), and the effects observed in such field tests would not be expected in groups of drilled shafts.
4. The focus of experimental research on group effects has been at relatively large displacements, as would be anticipated for extreme event loadings. The effects are less pronounced and P_m values are closer to unity at relatively small lateral shaft displacements of less than one inch.
5. Limited experimental data on groups of piles and drilled shafts loaded at a velocity comparable to extreme events such as seismic or vessel impact suggest that P_m values are similar to those obtained from static loading at similar displacement.

For general design of foundations composed of groups of drilled shafts, the P_m values provided in Table 11-1 are suggested.

TABLE 11-1 RECOMMENDED P-MULTIPLIER, P_M, VALUES FOR DESIGN BY ROW POSITION

Row Position	***Design P-multiplier, P_m***			
Pile Spacing (c-c)	3D	4D	5D	≥ 6D
Lead Row	0.7	0.85	1.0	1.0
2nd Row	0.5	0.65	0.85	1.0
3rd and higher Rows	0.35	0.5	0.7	1.0

11.5.2 Use of P-multiplier in Computer Codes

If the shaft heads are restrained in any way, moments will develop at the shaft heads that will cause the cap to rotate and to induce compressive and tensile loads in the shafts, such that the sum of the shaft-head moments is resisted by the sum of the push-pull couples in the shafts within the group and possibly partly by soil resistance against the cap. The cap rotation will also serve to relieve somewhat the moments applied to the shaft heads. The engineer can ignore this effect and design using the solutions from the single-shaft computer code, or he or she can use a computer code that considers all of the interactions among the shafts in the group, including this effect.

It should be noted that the p-multiplier approach described above is empirical and based on models which have been calibrated to experiments that are thought to represent typical foundation problems. As an alternative to the p-y method for very complex problems of soil-structure interaction, software now exists that will permit the nonlinear analysis of drilled shafts or groups of drilled shafts using the finite element method (FEM) with relative ease on a high-end PC or a workstation, for example ABAQUS (Hibbett et al., 1996). FEM analysis may be justified when the soil or rock conditions, foundation geometry or loading of the group is unusual.

11.5.2.1 Computer Codes for Analysis of Groups of Shafts

Hoit et al. (1997) describe FBPIER (now FBMultipier), a computer code that is capable of considering coupled effects of the drilled shafts and foundation cap in addition to much more complex three-dimensional group configurations, three-dimensional loading conditions, caps with flexibility, the soil resistance against the cap, and similar features. Ensoft, Inc. offer a similar code, called GROUP, that performs the calculations in two or three dimensions. Both codes have the capability of allowing for the consideration of lateral group action within the group through the use of p-multipliers, and both run in a user-friendly Windows environment. Space does not permit the description of these computer codes here; however, the reader is encouraged to obtain and review the literature cited above in preparation for the analysis of complex drilled shaft groups with lateral loads.

The values provided in Table 11-1 may be input into a computer code such as FBMultipier or GROUP so that row-specific modification of p-y curves are imposed on individual shafts within the group for the analysis.

The computer code DFSAP (available from the Washington State DOT) uses the strain wedge method to generate relationships of lateral soil resistance against drilled shaft foundations. This method estimates

group effects on the lateral soil resistance using overlapping passive soil wedges; accordingly, externally applied scaling effects as described in the previous section are not applied.

11.5.2.2 Analyses of Single Shafts for Group Model

Alternatively, the approach outlined by Brown and Bollmann (1993) may be used with multiple separate analyses of single shafts in order to compute the overall group response and the distribution of shear and moment to individual shafts within the group. The results from a computer analysis of a single drilled shaft can be used to compute a lateral load versus deflection response as indicated on Figure 11-11; at a given deflection, the total lateral resistance of the entire group will be the sum of the lateral resistances of the individual shafts at that deflection. Note that the variation in lateral load between drilled shafts in the group is less significant than the variation in p-multipliers between rows.

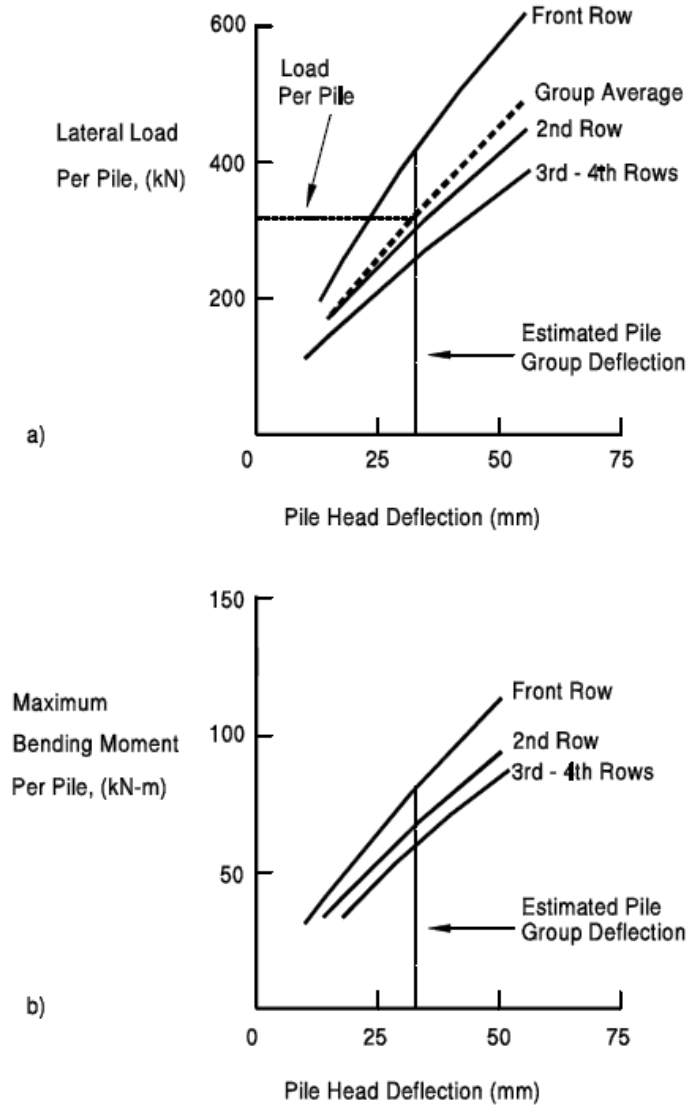


Figure 11-11 Example Plots of Lateral Load Response by Row Position. 25.4 min = 1 inch; 1 kN-m = 0.738 ft-kips

An alternative and simpler approach is to analyze the response of shafts in the group using an average P_m value for all of the shafts within the group. The average P_m value is computed by using a weighted average for all of the shafts based on the individual P_m values given in Table 11-1. Analyses and experimental data suggest (Brown et al., 2001) that this simpler approach:

1. captures the overall group stiffness to lateral loading with sufficient accuracy for design, given the numerous other uncertainties in real-world behavior
2. allows analyses of multi-directional loading with a single model without the need to adjust row-dependent P_m variables
3. can accommodate variation in maximum bending moment in any shaft within the group by using a simple overstress allowance above the maximum bending moment computed for the average shaft. For shafts at various center-to-center spacing, the maximum bending moment for any shaft within the group would be no greater than the computed maximum bending moment for the average shaft multiplied by the following:
    - 3D c-c, use $M_{max} = 1.2 \times M_{max,\ average}$
    - 4D c-c, use $M_{max} = 1.15 \times M_{max,\ average}$
    - 5D c-c, use $M_{max} = 1.05 \times M_{max,\ average}$

In general and for simplicity, all of the shafts within a group will be designed to include the same reinforcement. This is consistent with current practice, and is recommended to simplify construction and avoid any risk associated with installing the wrong reinforcement in a shaft within the group.

11.6 COMBINED LOADING AND COMPUTATION OF LOAD DISTRIBUTION TO GROUP

Efficient design of groups of drilled shafts requires analysis of group behavior to determine the distribution of forces to the individual shafts from combined loadings. An efficient group will distribute forces such that the resistance of all of the shafts in the group is utilized for the most critical load conditions.

For groups of shafts subjected to combined axial, lateral, and overturning forces, calculation of load distribution to the group may use one of the following approaches (in order of increased complexity):

- Simple static equilibrium (if the group can be modeled as a determinate frame)
- Elastic solution (model the individual shaft resistance with springs and the cap as a rigid body)
- Nonlinear solution (using a nonlinear computer code such as GROUP or FBPIER)

The simple static or elastic solutions are suitable for routine design, but may overestimate load concentrations to some individual shafts within the group for extreme event load cases where deflections are large. In such cases a nonlinear solution may be used to better represent the vertical, transverse, and rotational stiffness of the group, and more realistically model conditions at or approaching the geotechnical strength limit state condition.

The following sections provide an overview of the simple solutions that can be performed by hand, and a general description of the use of nonlinear computer solutions for shaft group problems.

11.6.1 Simple Static Equilibrium

For very simple group arrangements, it is possible to compute individual loads on the shafts within the group using equations of static equilibrium. This approach presumes a weighted average p-multiplier as described in Section 11.5.2.2. For purposes of this simple static analysis, the rotational restraint provided by the shaft head to rotation of the cap is ignored (inclusion of this component would make the problem statically indeterminate), and the moment applied to the cap is assumed to be resisted solely by shaft axial forces. Axial stiffness of all shafts in the group is assumed to be the same.

For example, consider a simple four shaft group in a 2 x 2 arrangement with a two-dimensional loading condition as shown on Figure 11-12. Shaft spacing, s, is 3D center to center (D = shaft diameter) and the cap has thickness, t, and weight, W_c. The lateral loading, Q_x, is in the x direction and the vertical loading, Q_z, in the z direction, with an applied overturning moment, M_{xz}, in the x-z plane. The shaft resistances are modeled as an axial resistance, R_{zi}, which may vary with position, and a horizontal resistance, R_x, which is assumed as equal for all shafts. The moment at the head of each shaft is ignored.

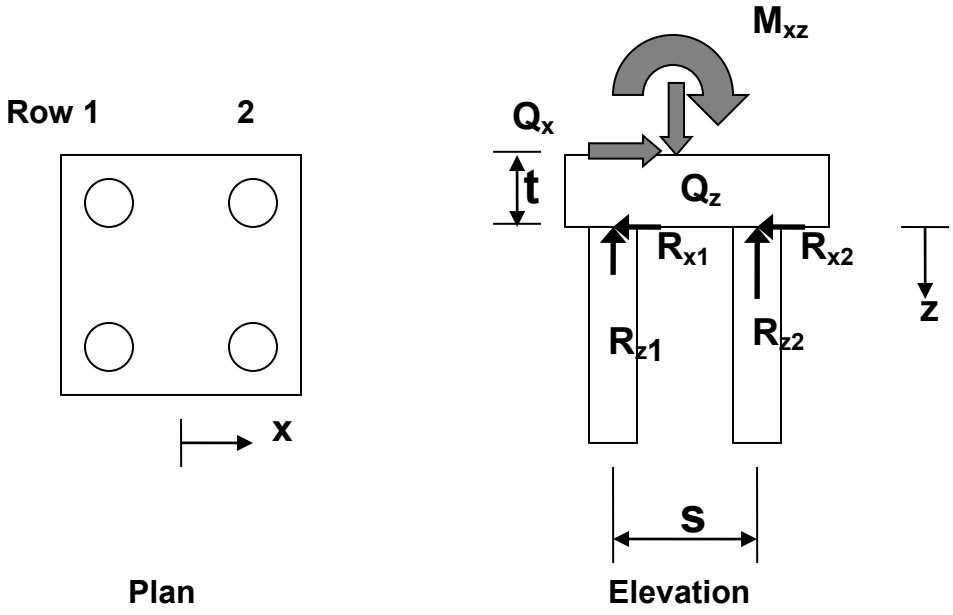


Figure 11-12 Simple Static Analysis of a 2 x 2 Group

For these simplifying assumptions, the resulting forces can be computed by statics. Force equilibrium in the x-direction:

$$Q_x = \sum R_x \; ; \qquad 11\text{-}15$$

Therefore, since there are four shafts in the group:

$$R_x = Q_x/4 \qquad 11\text{-}16$$

Since the axial stiffness of all the shafts is the same, force equilibrium in the z-direction is:

$$Q_z + W_c = \sum R_{zi}, = 2(R_{z1}) + 2(R_{z2}) \quad \text{11-17}$$

Rearranging Equation 11-17:

$$R_{z1} = (Q_z + W_c)/2 - R_{z2} \quad \text{11-18}$$

Summing moments about the top of row 1:

$$M_{xz} + (Q_z + W_c)(1.5D) + (Q_x)(t) = 2(R_{z2})(3D) \quad \text{11-19}$$

Therefore, Equation 11-19 can be rearranged to:

$$R_{z2} = [M_{xz} + (Q_z + W_c)(1.5D) + (Q_x)(t)]/6D \quad \text{11-20}$$

And Equation 11-20 can be substituted into Equation 11-18:

$$R_{z1} = (Q_z + W_c)/2 - [M_{xz} + (Q_z + W_c)(1.5D) + (Q_x)(t)]/6D \quad \text{11-21}$$

Consider the following example:

- 2 x 2 group of 4-ft diameter shafts (D=4 ft, s = 12 ft)
- Cap is 20 ft by 20 ft, 4 ft thick,
- W_c = (20)(20)(4)(0.15 k/cu.ft.) = 240 kips
- Q_z = 1500 kips
- Q_x = 120 kips
- M_{xz} = 1800 kip-ft

The forces on individual shafts are then:

R_{z2} = $[M_{xz} + (Q_z + W_c)(1.5D) + (Q_x)(t)]/6D$
= [1800 + (1500+240)(6) + (120)(4)]/24
= 530 k/shaft

R_{z1} = $(Q_z + W_c)/2 - [M_{xz} + (Q_z + W_c)(1.5D) + (Q_x)(t)]/6D$
= (1500+240)/2 - [1800 + (1500+240)(6) + (120)(4)]/24
= 340 k/shaft

R_x = $Q_x/4$ = 30 k/shaft (average)

Design for individual shafts would then be performed for an axial load demand of 530 kips and a lateral load demand of 30 kips. For the lateral analysis, the analysis of a single shaft would be performed using an average P-multiplier from Table 11-1:

$P_m = [2(0.7) + 2(0.5)]/4 = 0.6$

And the computed maximum moment in the shaft for the single shaft analysis would be multiplied by 1.2 to account for variation within the group (Section 11.5.2.2) for 3D center-to-center shaft spacing.

11.6.2 Simple Elastic Solution

For most group arrangements, it is possible to compute group deflections and individual loads on the shafts within the group by hand using a simple elastic solution. With this approach, the axial, transverse, and rotational stiffness of each shaft is modeled as a simple elastic spring. The cap is typically assumed to deform as a rigid body. The shafts may be modeled each with different lateral stiffness or by using the same stiffness based on analyses with a weighted average p-multiplier as described in Section 11.5.2.2. The rotational restraint provided by the shaft head to rotation of the cap can be included in the model, although the rotational stiffness from this source is typically much smaller than the stiffness resulting from the axial shaft forces times their moment arm.

For example, consider a simple six shaft group in a 2 x 3 arrangement with a two-dimensional loading condition as shown on Figure 11-13. Shaft spacing, s, is 3D center to center (D = shaft diameter) and the cap has thickness t and weight, W_c. The lateral loading, Q_x, is in the x direction and the vertical loading, Q_z, in the z direction, with an applied overturning moment, M_{xz}, in the x-z plane.

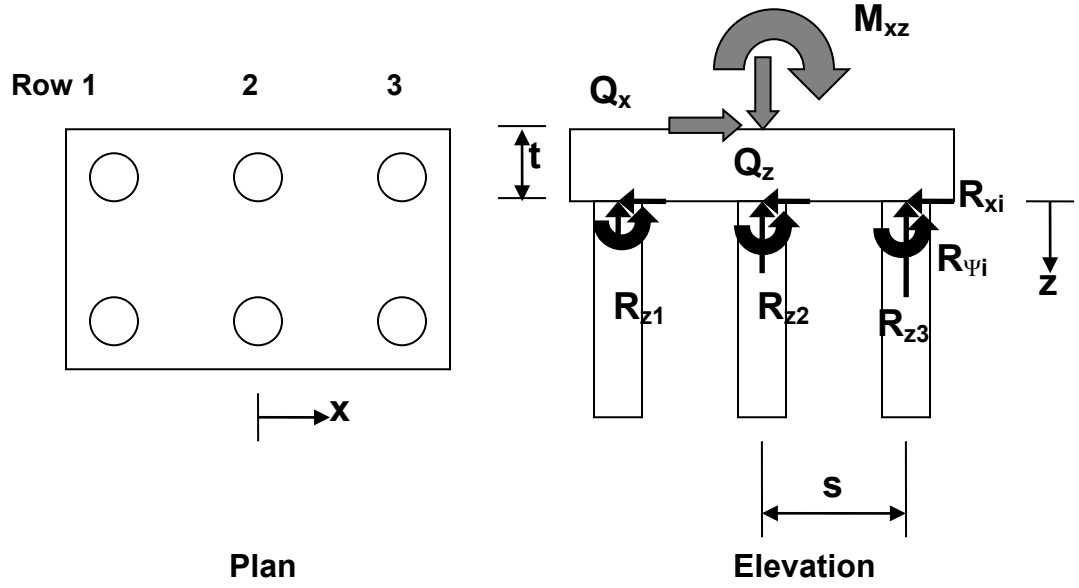


Figure 11-13 Simple Elastic Solution of a 2 x 3 Group

The shaft forces are modeled as an axial resistance, R_{zi}, and a horizontal resistance, R_{xi}, each of which may vary with position and are assumed to be a linear function of shaft head displacements, Δ_{zi} and Δ_{xi}, in the z and x directions, respectively, in accordance with the stiffnesses in the x and z directions:

$$R_{xi} = k_{xi}\Delta_{xi} \quad 11\text{-}22$$

$$R_{zi} = k_{zi}\Delta_{zi} \quad 11\text{-}23$$

The moment at the head of each shaft, $R_{\Psi i}$, is assumed to be a linear function of cap rotation, Ψ_{zz}, in accordance with the rotational stiffness of an individual shaft:

$$R_{\Psi i} = k_{\Psi i}\,\Psi_i \quad 11\text{-}24$$

The translational and rotational stiffness of the group about the bottom center of the cap (center of coordinate system, located here for convenience) can be defined by summing the individual shaft stiffnesses as outlined below. The equations shown are for simple groups composed of vertical drilled shafts only; battered shafts (rarely used and not recommended) require consideration of the x and z components of the axial and transverse shaft stiffness due to the difference in the local (shaft) axes and the global axes.

For a group of vertical shafts, the total resistance of the group in the x and z directions is:

$$R_{Gx} = \sum k_{xi}\Delta_{xi} \quad 11\text{-}25$$

$$R_{Gz} = \sum k_{zi}\Delta_{zi} \quad 11\text{-}26$$

and the moment resistance of the group in the z-x plane is:

$$R_{G\Psi} = \sum k_{\Psi i}\,\Psi_i + \sum k_{zi}\Delta_{zi}x_i \quad 11\text{-}27$$

where x_i is the x-coordinate of the top of the shaft as indicated on Figure 11-13.

Note that the simple solution described above ignores cross-coupling stiffness terms for individual shafts; i.e., the cross terms between rotation and translation of an individual shaft at the head. For typical shaft groups, these terms would be relatively insignificant and small compared to the accuracy with which other parameters are known.

Individual shaft displacements and rotations are related to the group displacements and rotations as follows:

$$\Delta_{xi} = \Delta_{Gx} \quad 11\text{-}28$$

$$\Delta_{zi} = \Delta_{Gz} + \Psi_G\,x_i \quad 11\text{-}29$$

$$\Psi_i = \Psi_G \quad 11\text{-}30$$

Substituting Equations 11-28 through 11-30 into Equations 11-25 through 11-27 provides:

$$R_{Gx} = [\sum k_{xi}]\Delta_{GX} \qquad 11\text{-}31$$

$$\begin{aligned} R_{Gz} &= [\sum k_{zi}]\{\Delta_{GZ} + \Psi_G x_i\} \\ &= [\sum k_{zi}]\{\Delta_{GZ}\} + [\sum k_{zi}x_i]\{\Psi_G\} \end{aligned} \qquad 11\text{-}32$$

$$\begin{aligned} R_{G\Psi} &= \sum k_{\psi i}\,\Psi_G + \sum k_{zi}(\Delta_{GZ} + \Psi_G x_i)x_i \\ &= \sum k_{zi}(\Delta_{GZ})x_i + \sum k_{\psi i}\,\Psi_G + \sum k_{zi}(\Psi_G x_i)x_i \\ &= [\sum k_{zi}x_i]\,(\Delta_{GZ}) + [\sum k_{\psi i} + \sum k_{zi}(x_i)^2]\,\Psi_G \end{aligned} \qquad 11\text{-}33$$

The solution is obtained by substituting the applied forces for the resistance (R) terms in Equations 11-31 through 11-33 and solving the three simultaneous equations for the group displacements. These equations can be expressed in matrix form as:

$$\begin{bmatrix} \sum k_{xi} & 0 & 0 \\ 0 & \sum k_{zi} & \sum k_{zi}x_i \\ 0 & \sum k_{zi}x_i & \sum k_{\psi i} + \sum k_{zi}(x_i)^2 \end{bmatrix} \begin{Bmatrix} \Delta_{GX} \\ \Delta_{GZ} \\ \Psi_G \end{Bmatrix} = \begin{Bmatrix} F_{Gx} \\ F_{Gz} \\ M_G \end{Bmatrix} \qquad 11\text{-}34$$

or, alternatively:

$$\begin{bmatrix} K_X & 0 & 0 \\ 0 & K_Z & K_{Z\Psi} \\ 0 & K_{\Psi Z} & K_\Psi \end{bmatrix} \begin{Bmatrix} \Delta_{GX} \\ \Delta_{GZ} \\ \Psi_G \end{Bmatrix} = \begin{Bmatrix} F_{Gx} \\ F_{Gz} \\ M_G \end{Bmatrix} \qquad 11\text{-}35$$

where the stiffness matrix terms are as indicated.

The force vectors are the net resultant applied forces at the center of the coordinate system as used to develop the equations (bottom center of the cap). Note that since there are no batter shafts, then the first equation (top row) is uncoupled from the other two and the calculations are simplified.

The following example demonstrates, consistent with the illustration on Figure 11-13:

- 2 x 3 group of 4-ft diameter shafts (D=4 ft, s = 12 ft)
- Cap is 20 ft by 32 ft in plan, 5 ft thick,
- W_c = (20)(32)(5)(0.15 k/cu.ft.) = 480 kips
- Q_z = 2500 kips
- Q_x = 180 kips
- M_{xz} = 1800 kip-ft

Analyses of an individual shaft subject to lateral loading were performed using LPILE with a full moment connection to the cap assumed at the top of the shaft. For simplicity, the LPILE analyses have been performed using an average p-multiplier (Table 11-1):

$\boldsymbol{P_m}$ = [2(0.7) + 2(0.5) + 2(0.35)]/6 = 0.52

The analyses determined that an individual drilled shaft would deflect approximately 0.25 inches for a lateral force at the top of 30 kips and the shaft head restrained against rotation (fixed head condition). Therefore, the individual shaft stiffness against horizontal translation is:

k_{xi} = 30/0.25 = 120 kips per inch

and the group stiffness term, K_x is:

$$K_X = \sum k_{xi} = 120(6) = 720 \text{ kips per inch}$$

The analyses determined that an individual drilled shaft would rotate approximately 0.002 in/in for an overturning moment at the top of 100 kip-ft. Therefore the individual shaft stiffness against rotation is:

$k_{\Psi i}$ = 100/0.002 = 50,000 kip-ft per inch/inch

Analyses of axial deformation under short term loading of individual shafts following the procedures outlined in Chapter 10 indicate that an axial deformation of around 0.125 inches is anticipated for an axial load on an individual shaft of 400 kips. Therefore, the individual shaft stiffness against vertical displacement is:

k_{zi} = 400/0.125 = 3200 kips per inch

The remainder of the group stiffness terms are:

$$K_Z = \sum k_{zi} = 3200(6) = 19{,}200 \text{ kips per inch}$$

$$K_{Z\Psi} = K_{\Psi Z} = \sum k_{zi} x_i = 3200(2)(-24\text{ft}) + 3200(2)(+24\text{ft}) = 0 \text{ k-ft/in.}$$

$$K_\Psi = \sum k_{\Psi i} + \sum k_{zi}(x_i)^2_i = (50{,}000)(6) + 3200(2)(-24)^2 + 3200(2)(24)^2$$
$$= 300{,}000 + 3{,}686{,}400 + 3{,}686{,}400 = 7{,}672{,}800 \text{ k-ft/(in/in)}$$

Note that the rotational stiffness contribution ($\sum k_{\Psi i}$) related to the individual shaft head rotational stiffness is more than an order of magnitude smaller than the rotational stiffness contribution of the axial force couples.

The forces on the group are as follows.

Q_x = 180 kips

$Q_z + W_c$ = 2980 kips

$M_{xz} + Q_x(t)$ = 1800 + 180(5) kip-ft = 2700 kip-ft

The terms in Equation 11-35 can now be shown as:

$$\begin{bmatrix} 720 & 0 & 0 \\ 0 & 19{,}200 & 0 \\ 0 & 0 & 7{,}672{,}800 \end{bmatrix} \begin{Bmatrix} \Delta_{Gx} \\ \Delta_{Gz} \\ \Psi_G \end{Bmatrix} = \begin{Bmatrix} 180 \\ 2{,}980 \\ 2{,}700 \end{Bmatrix}$$

and the solution for the displacement vector is:

Δ_{GX} = 180/720 = 0.25 inches

Δ_{GZ} = 2980/19,200 = 0.155 inches

Ψ_G = 2700/7,672,800 = 0.000352 inch/inch

The individual shaft displacements can be determined using Equations 11-28 and 11-29:

$\Delta_{xi} = \Delta_{Gx}$ = 0.25 inches, all shafts Rows 1-3

$\Delta_{zi} = \Delta_{Gz} + \Psi_G x_i$

Δ_{z1} = 0.155 in + 0.000352 (-24 ft)(12 in/ft) = 0.054 inches, Row 1

Δ_{z2} = 0.155 in + 0.000352 (0)(12 in/ft) = 0.155 inches, Row 2

Δ_{z3} = 0.155 in + 0.000352 (24 ft)(12 in/ft) = 0.256 inches, Row 3

The individual shaft forces can be determined using Equations 11-22 and 11-23:

$R_{xi} = k_{xi}\Delta_{xi}$ = (120 k/in)(0.25 in) = 30 kips, all shafts Rows 1-3

$R_{zi} = k_{zi}\Delta_{zi}$

R_{z1} = (3200 k/in)(0.054 in) = 173 kips/shaft, Row 1

R_{z2} = (3200 k/in)(0.155 in) = 496 kips/shaft, Row 2

R_{z3} = (3200 k/in)(0.256 in) = 819 kips/shaft, Row 3

So, the maximum axial load per shaft for design is 819 kips. The lateral shaft design should be completed using an analysis of a single shaft with a lateral force of 30 kips and a rotation of 0.000352 at the shaft head, and a p-multiplier, $\boldsymbol{P_m}$ = 0.52. The maximum computed bending moment in the shaft should be multiplied by 1.2 to account for variation in load distribution to the shafts within the group.

11.6.3 Nonlinear Computer Solution

For three dimensional loadings of a group of drilled shafts, a computer code is very useful in performing the length computations needed to determine the distribution of forces to the individual drilled shafts. There are a number of available computer codes which can be used for this purpose, and the use of a computer can provide a powerful analysis tool so that the design engineer can quickly optimize the layout of individual drilled shafts within a group for maximum efficiency.

Computer codes such as those described in Section 11.5.2.1 include the capability to account for nonlinear effects in the axial and lateral soil resistance, nonlinear flexural response of the drilled shaft to bending moments, and group effects on the lateral soil resistance. Some codes may include the lateral soil resistance provided by an embedded cap. However, many of the issues related to group effects on axial resistance described in Section 11.4 (particularly those related to settlement of groups of drilled shafts) are not addressed in typical computer codes used for structural analysis of bridge foundations. Powerful three dimensional nonlinear finite element codes used in geotechnical engineering may be useful for computation of settlement of groups of drilled shafts, but the use of these tools for bridge foundations is not common practice. Settlement does not often control the design of bridge foundations as might be the case for a high rise building or a pile-supported embankment.

The capabilities of computer software are rapidly evolving, and many general purpose codes used for structural analysis are incorporating features for nonlinear analysis of foundation elements. A model which incorporates multiple piers and multiple nonlinear foundations can be very useful in the analysis of extreme event loads such as vessel impacts or a train derailment, because a portion of a large transient load which is applied to one foundation can be redistributed through the structure to other foundations. A more efficient and cost-effective design often results from the use of these more sophisticated analytical tools.

A description of the capabilities and characteristics of individual computer codes is beyond the scope of this manual, and the reader should refer to the documentation to understand the features of specific software used for analysis.

11.7 SUMMARY

This chapter provided an overview of the considerations for the design of groups of drilled shafts. Although drilled shafts are often used to greatest advantage by utilizing a single drilled shaft at each individual column locations to avoid the need for a cap, extremely large diameter drilled shafts needed for some single shaft foundations may be uneconomical or inefficient, or pose unacceptable construction risks. Groups of drilled shafts are seen to provide capabilities to support much larger foundation loads, particularly where large overturning moments are applied. They often also can reduce foundation displacement and rotation due to their stiffer response to applied loads. The effects of spacing and group dimensions on axial and lateral soil resistance are described in this chapter, along with methods of analysis used to incorporate group effects. The analytical methods described in this chapter demonstrate how the analyses may be performed to determine the distribution of loads to the drilled shafts within a group and thereby design the layout of the group and the individual drilled shafts within the group for the computed forces.

In many cases, the large controlling load cases which may necessitate the use of a group of drilled shafts are derived from consideration of extreme event loadings. Chapters 9 and 10 provide discussions of the design of drilled shafts for extreme events.

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CHAPTER 12
STRUCTURAL DESIGN

12.1 INTRODUCTION

In Chapter 1 a drilled shaft is defined as a cast-in-place, reinforced concrete *structural element* constructed in a stable hole excavated in the ground. When used to support a structure such as a bridge, a drilled shaft is an integral load-bearing component of the structural system. When used in earth retention or slope stabilization applications, the drilled shaft is also part of a structural system, but with a significant portion of the load coming from ground forces. In all cases, the drilled shaft must be designed to withstand the structural demands imposed by the supported structure and/or ground forces, and to limit deformations required by service limit state criteria. To satisfy this part of the design, a drilled shaft is therefore treated in the same manner as other load-bearing reinforced concrete elements; however, consideration must also be given to the unique constructability requirements of drilled shafts, all of which are directly related to the fact that construction takes place in an excavation made in the ground.

Methods for structural design of drilled shafts presented in this chapter follow the provisions of Section 5 of the AASHTO LRFD 2017 Bridge Design Specifications, 8th Edition (AASHTO, 2017a) and are consistent with recommended practices for drilled shaft constructability as emphasized throughout this manual and also presented in Section 5 "Drilled Shafts" of the AASHTO LRFD Bridge Construction Specifications, 4th Edition (AASHTO, 2017b). In this chapter, the terms 'AASHTO design code' and 'AASHTO construction code' are used when referring to the 2017 LRFD bridge design and construction specifications, respectively.

Structural design is identified as Block 12 in Figure 8-1, the outline of the overall design process. Design of drilled shafts is an iterative process, and at various stages revisions may be required to the structural design to improve constructability, or to address changes in shaft diameter or length in response to revised loads, updated subsurface information, or as a result of revisions to the geotechnical design.

Figure 12-1 outlines the general steps involved in structural design of drilled shafts. The process is carried through for each trial design under consideration. In most cases, trial designs have already been advanced to the level of establishing trial depths and diameters based on geotechnical design considerations for axial and lateral loading. The objective of final design, including structural design, is to produce a set of plan drawings detailing the properties and geometry of the reinforced concrete shafts that will enable a qualified contractor to install the drilled shafts in accordance with the design intent. The design shown on the drawings must be supported by calculations demonstrating that the LRFD criterion is satisfied for all applicable limit states and load cases. Design of reinforced concrete members as covered in Section 5 of the AASHTO design code requires consideration of axial, shear, and moment demands. Under combined axial and moment loads, drilled shafts are treated as reinforced-concrete beam-columns and are analyzed in terms of the axial load (P) and moment (M) interaction diagram (P-M diagram). It is assumed that the structural designer is a licensed engineer with fundamental understanding of reinforced concrete behavior and design as presented in governing codes and standard textbooks on the subject (*e.g.*, Fanella 2016, Wight 2016, others).

For each trial design:

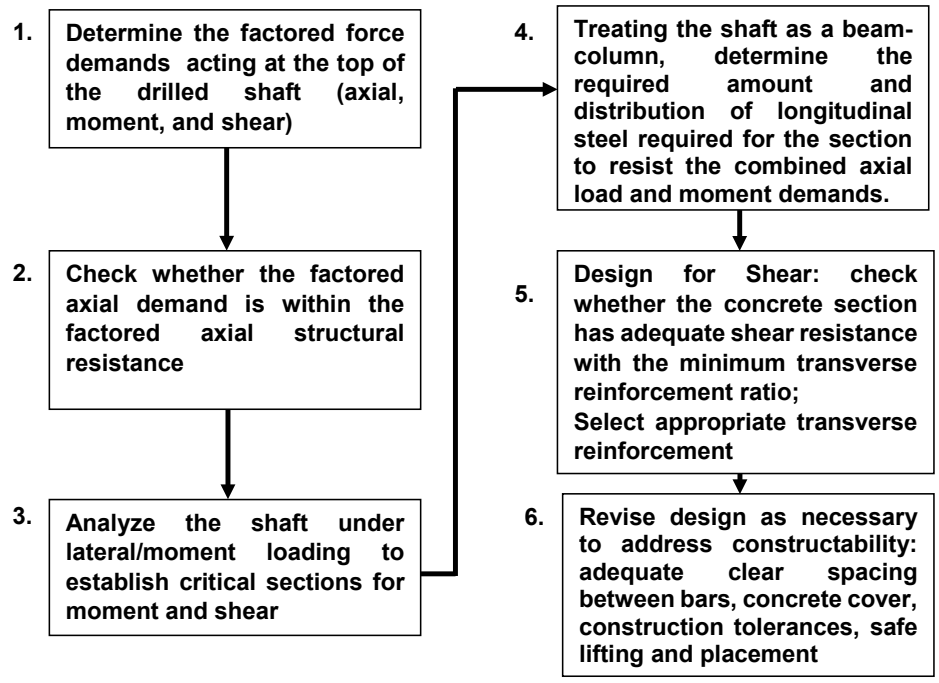


Figure 12-1 Simplified Flow Chart for Structural Design.

The LRFD design criterion for structural design of drilled shafts, as for all structural elements of the bridge or other structure, is that the factored structural demand may not exceed the factored structural resistance, or:

$$\Sigma\eta_i\gamma_i Q_i \leq \phi R_n = R_r \qquad 12\text{-}1$$

where:

γ_I = load factor: a multiplier applied to force effects

φ = resistance factor: a multiplier applied to nominal resistance, as specified in AASHTO

η_i = load modifier: a factor relating to ductility, redundancy, and operational classification

Q_i = force effect

R_n = nominal resistance

R_r = factored resistance: φR_n

12.2 MATERIAL REQUIREMENTS

Properties of structural materials used in drilled shafts are discussed in Chapters 5 (steel casing), 6 (reinforcing steel), and 7 (concrete). Unless otherwise noted, materials and material properties should be in accordance with the provisions of the most up-to-date version of the AASHTO LRFD design and construction codes. Some of the relevant structural considerations for these material properties are summarized in the following sub-sections.

12.2.1 Concrete

Drilled shafts are generally designed with concrete having a specified compressive strength, f_c', in the range of 3.5 ksi to 5.0 ksi, although the design methods in the AASHTO design code are generally applicable for design compressive strengths up to 10.0 ksi. Note that the AASHTO design code (5.4.2.1) states that concrete with compressive strengths used in design below 2.4 ksi should not be used in structural applications.

Concrete for drilled shafts should be normal weight. The modulus of elasticity (E_c) for normal weight concrete with water-to-cement ratio of w_c = 0.145 and design compressive strengths up to 10 ksi can be approximated by Equation 12-2:

$$E_c = 1820\sqrt{f_c'} \quad \text{with } E_c \text{ and } f_c' \text{ in ksi} \qquad \text{(AASHTO C5.4.2.4.-1)} \qquad 12\text{-}2$$

Section 5.4.2 of the AASHTO design code and Chapter 7 of this manual provide additional information on properties of concrete used for drilled shafts.

12.2.2 Reinforcing Steel

Chapter 6 identifies the ASTM and AASHTO designations and corresponding properties of steel used in concrete reinforcing (see Table 6-1). The most common reinforcing steel for drilled shafts has been AASHTO M31 (ASTM A615) Grade 60, with a minimum yield strength (f_y) of 60 ksi. However, there is a growing trend in the use of higher grade steels for drilled shaft reinforcing. The current AASHTO design code allows the use of reinforcing steel up to Grade 100 (f_y = 100 ksi). Using higher strength bars reduces the amount of steel required, thus reducing cage weights and providing greater clear spacing between bars, both of which are advantageous for constructability. Use of reinforcing steel with yield strengths less than 60 ksi is not recommended and should be used only with the approval of the owner.

If rebar welding is under consideration, steel conforming to ASTM A706, "Low Alloy Steel Deformed Bars for Concrete Reinforcement," should be considered.

The modulus of elasticity, E_s, for reinforcing steel can be assumed to be 29,000 ksi.

Section 5.4.3 of the AASHTO design code and Chapter 6 of this manual provide additional information on drilled shaft reinforcing steel. Note that this manual does not address structural design of drilled shafts with a steel core used as reinforcement, for example a wide flange steel beam or steel pipe. These are more common in drilled shaft secant pile walls and some building applications but are not typically used in drilled shafts for bridge foundations. Section 6.9.5 of the AASHTO design code addresses structural design of composite members including concrete-encased steel shapes.

12.2.3 Casing

Steel for permanent casing should generally conform to the values shown in Table 12-1. The modulus of elasticity, E, for steel casings can be assumed to be 29,000 ksi.

Casing thickness should be shown in the contract documents as "minimum," The minimum thickness of casing should be that required for reinforcement or for strength required during installation. The latter is a function of both the site conditions and the installation method and equipment. The AASHTO design code requires the contractor to furnish casings of greater than the design minimum thickness, if necessary, to accommodate the contractor's choice of installation equipment. Readers are referred to Chapter 5 for more information on this topic.

TABLE 12-1 MINIMUM YIELD STRENGTHS FOR PERMANENT STEEL CASING

Standard	Minimum Yield Strength (f_y)
ASTM A36	36 ksi
ASTM A242	50 ksi for thickness ≤ ½" 46 ksi for ¾"< thickness ≤ 1½" 42 ksi for 1½"< thickness ≤ 4"
ASTM A252 Grade 2	36 ksi
ASTM A252 Grade 3	45 ksi

12.3 CODE-SPECIFIED LIMITS ON STEEL REINFORCEMENT

12.3.1 Longitudinal Steel

Section 5 of the AASHTO design code limits the amount of longitudinal steel (ρ_s) allowed in a reinforced concrete structural member to between 1% and 8% of the gross cross-sectional area. These limits are intended to assure ductile behavior of reinforced concrete under the anticipated loading conditions and for all limit states. The code further restricts the upper limit to 6% for seismic zone 2 and to 4% for seismic zones 3 and 4. From a practical standpoint, drilled shafts are typically designed with longitudinal reinforcement ratios between 1% and 2%. At higher steel percentages, cage congestion that restricts the ability of fluid concrete to pass between bars becomes a constructability challenge. In higher seismic zones, the calculated longitudinal steel percentages may be higher, approaching the 4% upper limit in some cases. To accommodate designs with these higher steel percentages, the designer should work closely with engineers knowledgeable of the means and methods that will allow concrete to be placed reliably, such as special concrete mix designs, or modify the design by using larger bars, bundling of bars, or possibly increasing shaft diameters to reduce rebar congestion.

The minimum amount of longitudinal reinforcement for compression members is given by:

$$\frac{A_s f_y}{A_g f_c'} \geq 0.135 \qquad \text{(AASHTO 5.6.4.2-3)} \qquad 12\text{-}3$$

where: A_s = area of longitudinal reinforcement (in^2)
f_y = yield strength of longitudinal steel reinforcement (ksi)
A_g = gross area of concrete cross section (in^2)
f_c' = compressive strength of concrete for use in design (ksi)

Where the unfactored permanent loads do not exceed $0.4A_g f_c'$, the reinforcement ratio given by Eq. 12-3 need not be greater than 1.5%. Furthermore, the minimum longitudinal reinforcement area in portions of the shaft that behave as a column (i.e., the portion above the ground surface) should be not less than 1% of the gross concrete area of the shaft. Below the section where the drilled shaft behaves as a column (*i.e.*, the section that is laterally supported) nominal longitudinal reinforcement may be provided. However, 0.5% of the gross concrete area of the drilled shaft is suggested as a practical minimum.

Longitudinal reinforcing bars should be evenly distributed among not less than 6 bars in a circular arrangement. The minimum size of longitudinal bars is No. 5 (AASHTO 5.6.4.2).

12.3.2 Transverse Steel

Article 5.7.2.3 of the AASHTO design code states that "transverse reinforcement shall be provided" where:

$$V_u > 0.5\varphi V_c \qquad \text{(AASHTO 5.7.2.3-1)} \qquad 12\text{-}4$$

In other words, where the factored shear demand, V_u, exceeds one half of the factored shear resistance provided by the concrete portion of the drilled shaft cross section (φV_c), a minimum amount of transverse steel is required. The required minimum is given in Article 5.7.2.5:

$$A_v \geq 0.0316\,\lambda\sqrt{f'_c}\,\frac{b_v s}{f_y} \qquad \text{(AASHTO 5.7.2.5-1)} \qquad 12\text{-}5$$

where: A_v = area of transverse reinforcement within distance s (in^2)
λ = concrete density modification factor (equals 1.0 for normal weight concrete)
f_c' = compressive strength of concrete for use in design (ksi)
b_v = width of web; can be taken as drilled shaft diameter
s = spacing of transverse reinforcement (in.)
f_y = yield strength of transverse reinforcement (ksi)

For sections where the factored shear demand falls below the level given by Equation 12-4, the minimum amount of transverse steel given by Equation 12-5 no longer applies; however drilled shaft cages require transverse steel for handling and safety as described in Chapter 6 (Rebar Cages). Article 5.7.2.6 establishes the maximum permissible spacing of transverse reinforcement as follows:

- If $v_u < 0.125\,f_c'$, then $s_{max} = 0.8d_v \leq 24.0$ in.
- If $v_u \geq 0.125\,f_c'$, then $s_{max} = 0.4d_v \leq 12.0$ in.

where: v_u = shear stress on the concrete = $V_u / (\varphi b_v d_v)$
d_v = effective shear depth (in.); see Figure C5.7.2.8-2 of AASHTO design code
b_v = effective shear width = drilled shaft outer diameter

Equations for nominal shear resistance provided by concrete (V_c) and reinforcing steel (V_s) are presented in Section 12.7 Structural Design Procedure, Design Step 5 (Design for Shear).

Section 5.11 of the AASHTO design code (Seismic Design and Details) provides additional restrictions and limits on transverse reinforcement for cast-in-place concrete piles (drilled shafts), which are applicable only to design for extreme event limit states. Detailed treatment of these provisions is beyond the scope of this manual, but the reader should be advised that some of these provisions lead to clear spacing between transverse reinforcement that do not conform to the recommended minimum clear spacings presented in Chapter 6 as best practice for constructability. In particular, Article 5.11.4.1.5, which applies to extreme event loading in Seismic Zones 3 and 4, states:

> "Transverse reinforcement for confinement shall be provided within piles in pile bents over a length extending from 3.0 times the maximum cross-sectional dimension below the calculated point of moment fixity to a distance larger than the greater of the maximum cross-sectional dimension or 18.0 in. above the mud line. Spaced not to exceed the lesser of the following: one-quarter of the minimum member dimension, or 4.0 in. center-to-center."

The above provision limiting maximum vertical spacing to 4 inches clearly contradicts the minimum clear spacing of 5 inches recommended for concrete flow. In those cases, the designer should work closely with engineers knowledgeable of the means and methods that will allow concrete to be placed reliably. Note that the California Amendments to AASHTO LRFD Bridge Design Specifications (2016) states:

> "For cast-in-place concrete piling, clear distance between parallel longitudinal, and parallel transverse reinforcing bars shall not be less than 5 in."

In the California Amendments, the language specifying exceptions to the 5-inch spacing for seismic requirements is deleted, meaning that the 5-inch minimum clear spacing is to be met for all seismic zones and all limit states. This practice, which recognizes the paramount importance of constructability, is highly recommended for consideration by all public agencies. Caltrans also promotes the use of large-diameter (No. 8) bundled hoops as a design method to meet the high seismic shear demand while also providing adequate clear spacing ($\geq$ 5 inches) needed for concrete to flow through the cage.

Example 12-1. Determine the minimum required bar size of Grade 80 spiral for a 5-ft diameter drilled shaft, design f_c' = 4 ksi. Assume a pitch s = 6 inches to ensure a minimum clear spacing of 5 inches.

Solution: By Eq. 12-5:

$$A_v \geq 0.0316\,\lambda\sqrt{f'_c}\,\frac{b_v s}{f_y} = 0.0316\,(1.0)\,4^{0.5}\,(60\text{in} \times 6\text{in})/80\text{ksi} = 0.28\ \text{in}^2$$

Minimum bar size meeting required cross-sectional area is No. 5, with $A_v = 0.31\ \text{in}^2$

Clear spacing = 6 inches – diameter (0.625 inch) = 5.38 inches > 5 inch

For pitch s = 6 inches, select No. 5, Grade 80 spiral.

12.4 CONCRETE COVER

AASHTO and ACI code requirements for cover when concrete is cast against soil or rock are 3.0 inches. However, taking into account construction tolerances, higher values of cover are recommended as drilled shaft diameter increases, which minimizes the potential for falling below 3 inches as a result of minor misalignment of the cage or deviations in the verticality of the excavation. Recommended minimum values of concrete cover to the primary (longitudinal) reinforcing steel (including rebars protected by epoxy coating) for drilled shafts are as follows:

- 3.0 inches for shafts ≤ 3'-0" diameter
- 4.0 inches for > 3'-0" but < 5'-0" diameter
- 6.0 inches for shafts ≥ 5'-0" diameter

The cover required for transverse reinforcement may be less than required for longitudinal bars by no more than 0.5 inch. Transverse reinforcement larger than 0.5-inch diameter would therefore necessitate greater cover than specified above for longitudinal bars.

The AASHTO design code allows modifications to the minimum covers given above, based on higher or lower values of water to cementitious material ratio (*w/cm*). The minimum values recommended above are for concrete with water-to-cementitious material ratios *(w/cm*) between 0.40 and 0.50. For *w/cm* values equal to or greater than 0.50, the cover requirements must be increased by a factor of 1.2. For *w/cm* values less than or equal to 0.40, the cover requirements may be decreased by a factor of 0.8. These modifications account for changes in concrete permeability as a function of *w/cm*. However, low values of *w/cm* reduce the ability of concrete to pass through the rebar cage. It is therefore not recommended to reduce cover requirements for drilled shafts to values less than those given above.

Centering devices are used to maintain the alignment of the steel reinforcing cages and maintain the required minimum concrete cover. Types of centering devices and their use for cage installation are described in Chapter 6.

12.5 DESIGN FOR AXIAL LOAD

12.5.1 Axial Compression

For some applications, the load demands transmitted to drilled shafts are predominately axial compression with zero to small moment and shear. Structurally, the drilled shaft can then be designed for axial compression only. Eccentricity of the axial load is ignored explicitly; however, the equation for nominal compressive resistance incorporates a reduction factor to account for minor eccentricities that occur as a result of construction tolerances.

Equation 12-6 provides the factored structural resistance of a reinforced concrete column subjected only to compressive axial load, for concrete compressive strengths up to 10 ksi.

$$P_r = \varphi P_n = \varphi\beta\left[0.85 f_c^{'}(A_g - A_s) + A_s f_y\right] \qquad \text{(AASHTO 5.6.4.4)} \qquad 12\text{-}6$$

where: P_r = Factored axial resistance of an axially loaded short column (drilled shaft)
P_n = Nominal axial resistance
φ = Resistance factor (see below)
β = Reduction factor: 0.85 for spiral reinforcement and 0.80 for tie reinforcement.
f_c' = Specified minimum compressive strength of concrete,
A_g = Gross area of section
A_s = Total area of longitudinal steel reinforcement
f_y = Specified yield strength of reinforcement

The resistance factor, φ is equal to 0.75 for compression-controlled sections with either spiral or circular ties (hoops) providing transverse reinforcement. An exception is for the case of extreme event seismic loading in Seismic Zones 2, 3 and 4, where φ is taken as 0.90 (see AASHTO 5.5.4.2, 5.11.3 and 5.11.4.1.2).

In executing a preliminary design to obtain the approximate cross-sectional area and longitudinal steel schedule, a reasonable percentage of steel between 1 and 4% (preferably 1 to 2%) of the gross column section area, A_g, can be assumed. If the drilled shaft is subjected to an axial load having an eccentricity larger than is permitted in the construction specifications for horizontal position of the drilled shaft, or if the calculated load demands include shear or moment, a lateral load analysis should be carried out. Note that an eccentric axial load will generate bending and therefore the shaft must be designed as a beam-column. Depending on the level of load eccentricity and the magnitudes of the lateral loads, the structural resistance for axial compression (alone) should be well in excess of the factored axial load so that the section will also be found to be safe under moment due to eccentricity only.

When the base of a drilled shaft is bearing on competent rock, the geotechnical base resistance provided by the rock may exceed the bearing strength of the concrete shaft (see discussion in Section 10.3.5.3). If the shaft reinforcement extends to the tip, Equation 12-6 provides the appropriate nominal compressive resistance, except that the resistance factor for bearing is 0.70. If the reinforcement does not extend to the tip, or if there is a likelihood that field adjustments will be made that result in deepening of the socket without increasing the reinforcement length, the concrete at the tip of the socket may be unreinforced (plain concrete). In that case, a check should be made of the factored bearing resistance for plain concrete, given by:

$$P_r = \varphi P_n = 0.70\,[0.85\,f_c'\,A_1\,m] \qquad \text{(AASHTO 5.6.5-2)} \qquad 12\text{-}7$$

where: A_1 = bearing area (cross-sectional area of the rock socket)
m = confinement modification factor: ≤ 2.0 for uniformly distributed bearing surface

For a socketed shaft bearing on competent rock the factor m can be taken equal to 2.0.

12.5.2 Tension Members

A drilled shaft subjected to uplift is treated as a tension member and the axial force is assumed to be resisted by the reinforcing steel only. The LRFD equation for structural strength in tension is:

$$P_r = \varphi P_n = \varphi\left(f_y A_{st}\right) \qquad \text{(AASHTO 5.6.6.1-1 and 5.8.2.4.1-1)} \qquad 12\text{-}8$$

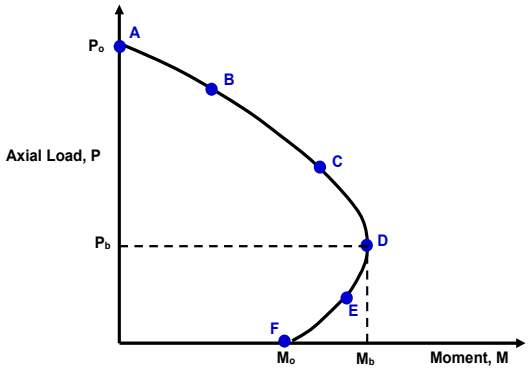
where: P_r = Factored axial resistance in tension
P_n = Nominal axial resistance in tension
φ = Resistance factor = 0.90
f_y = Specified yield strength of steel reinforcement
A_{st} = Total area of longitudinal steel reinforcement

12.6 INTERACTION DIAGRAMS FOR COMBINED AXIAL AND LATERAL LOADING

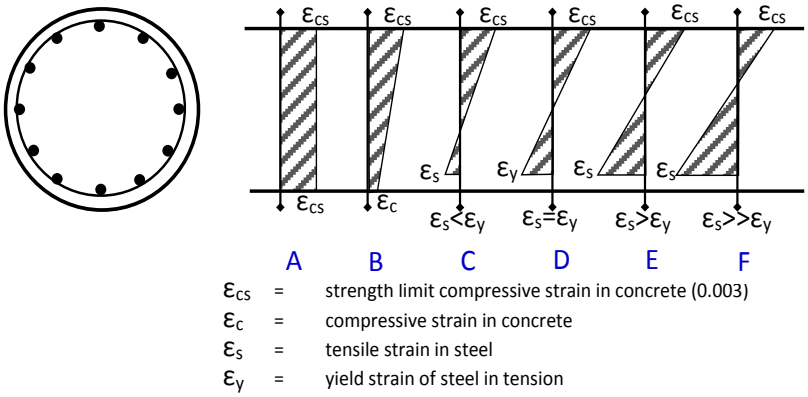
A reinforced concrete member subjected to bending is analyzed according to conventional beam theory based on Bernoulli's hypothesis that 'plane sections remain plane,' meaning that there is a linear variation of axial strain in the cross section. It is further assumed that axial loading results in a uniform distribution of strain in the cross section. When the two are superposed (axial and bending), each point on the curve shown in Figure 12-2(a) represents the strength limit of the section corresponding to the combined limiting values of axial load (P) and bending moment (M), referred to as a 'P-M interaction diagram'. In Figure 12-2(b), the distribution of strain is shown for varying combinations of axial load (P) and bending moment (M) corresponding to points A through F labeled on the interaction diagram of Figure 12-2(a). The criteria for defining the strength limit, in terms of limiting values of strains, are summarized as follows.

- Point A on the curve corresponds to pure axial compression loading, with zero moment, and results in a uniform distribution of compressive strain, ε_c. The strength limit is defined as the axial load that results in the strain causing crushing of the concrete, ε_{cs}. This limiting value of strain in the concrete is 0.003.
- At Point B the strength limit is reached at a lower magnitude of axial load combined with a small amount of bending moment. The strain distribution in the cross section is no longer uniform. The top-fiber strain reaches the value of ε_{cs} whereas the bottom-fiber strain is reduced, but is still compressive if the moment is not large.
- Point C represents a condition where increased bending moment generates tension, which is taken by the steel reinforcement, assuming that the concrete is cracked and cannot resist tension. If tension in the steel is not sufficient to cause yielding, the section strength is still governed by crushing strain in the concrete.
- At Point D, the strength-limit strain ε_{cs} in the concrete and tensile yield strain ε_y, in the steel are reached simultaneously. This is known as the balanced condition, and M_b and P_b are the moment and axial load resistances of the section at the balanced condition. At any strength limit combination between points A and D on the curve, failure is caused by crushing in the concrete before the steel yields, and is therefore described as being 'compression controlled.'
- Point E corresponds to the condition in which tensile yielding in the steel can occur under a smaller bending moment than at the balanced condition if the compressive axial load is of lower magnitude. Since the axial load is less, the steel yields before the strength-limit concrete strain, ε_{cs} is reached and this strength limit is described as 'tension controlled.'
- At Point F the section is subjected to bending moment only (M_o), and failure occurs well after the steel yields. With further bending, the concrete compressive strain reaches ε_{cs}.

The P-M interaction diagram for a given cross section depends on the shaft diameter, concrete strength (f_c'), layout and diameters of the longitudinal bars, and the steel reinforcement yield strength (f_y). For a drilled shaft with permanent casing, the P-M interaction diagram can also incorporate the contribution of the steel casing to the section strength based on the assumption that there is strain compatibility between the casing and concrete, *i.e.*, plane sections remain plane. For a given set of geometric and material properties, the equations to calculate the combined nominal values of axial load (P) and moment (M) that define the strength limit of the section can be found in any standard textbook on reinforced concrete design. However, in practice, the nominal interaction diagram is typically generated using commercially available software, which provides a convenient way to evaluate multiple trial designs. Application of P-M interaction diagrams to the structural design of drilled shafts is described in the next section.



(a) P-M interaction diagram for circular reinforced concrete section



(b) Member cross section and distribution of strain corresponding to Points A – F of interaction curve

Figure 12-2 Interaction diagram and strain distribution for a reinforced concrete column

12.7 STRUCTURAL DESIGN PROCEDURE

Figure 12-1 outlines six general steps involved in structural design of drilled shafts. Each step is presented below in more detail with illustrative examples.

12.7.1 Determine Factored Force Demands at Top of Shaft

For a bridge or other structure, this information is obtained from the structural design engineer. As described in Chapter 8, a structural model of the bridge is analyzed under the factored load combinations corresponding to each applicable limit state. Foundation supports can be modeled in several ways, including as springs or using an equivalent depth of fixity. The foundation support reactions are resolved into axial (P), shear (V), and moment (M) components and taken as the top-of-foundation force demands. Shear and moment may be further resolved into two mutually perpendicular components, for example corresponding to the longitudinal and transverse bridge directions.

It is important for the drilled shaft structural designer to be well informed on which limit states impose the maximum load demands on the drilled shafts and to design for the most critical cases. This requires good communication between the bridge structural engineer and the foundation designer.

12.7.2 Check Trial Design for Axial Resistance

For the trial shaft diameter, concrete design strength, and longitudinal reinforcement layout and yield strength, check the factored axial force (P) against the factored axial structural resistance $\varphi(P_n)$ using Equation 12-6 (compression) or Equation 12-8 (tension). If the factored demand exceeds the factored resistance, modify the trial design to increase the resistance. This can be accomplished by increasing any one of the structural components that contributes to axial strength, which includes the amount or grade (yield stress) of the longitudinal steel reinforcement, the diameter of the shaft, and/or the concrete strength.

12.7.3 Analyze the Shaft or Shaft Group Under Lateral/Moment Loads

The objective of this step is to determine the locations and magnitudes of maximum shear and moment demand. The drilled shaft(s) are then designed structurally to meet these demands. Methods for generating shear and moment versus depth diagrams are presented in Chapter 9 (Design for Lateral Loading). The most widely used approach in practice is the *p-y* curve method. Solutions can be obtained using commercially available software, for single drilled shafts or for groups of drilled shafts.

Loads applied at the top of the shaft are those obtained in Step 1 and should be the factored demands, consisting of axial, shear, and moment components. Appropriate boundary conditions can be imposed at the top of the shaft(s), for example free-head where a single shaft supports a single column, or fixed-head conditions where multiple drilled shafts are anchored into a cap or footing.

The maximum values of axial load, shear, and moment demand do not typically occur at the same locations over the depth of the shaft. To illustrate, consider the drilled shaft cross section of Example 12-1, with diameter = 5 ft and f_c' = 4 ksi. A drilled shaft with this cross section is being used to support a single column of a bridge. The subsurface profile, factored force demands at top-of-shaft (Step 1), and trial section properties are shown in Figure 12-3. The trial drilled shaft is 47 ft deep and socketed 5 ft into rock. The trial section design consists of eighteen No. 11 bars, Grade 80, with 6-inch concrete cover. Based on the *p-y* method of analysis, the resulting shear and moment diagrams are shown in Figure 12-4 (Step 3). Maximum shear demand V_{max} = 247 kips and occurs at a depth of 18 ft below top-of shaft. Maximum moment demand M_{max} = 4,400 k-ft and occurs at 6 ft below top-of-shaft, which is slightly more than one shaft diameter.

These results will be used in Steps 4 and 5 to illustrate design concepts for flexure and shear.

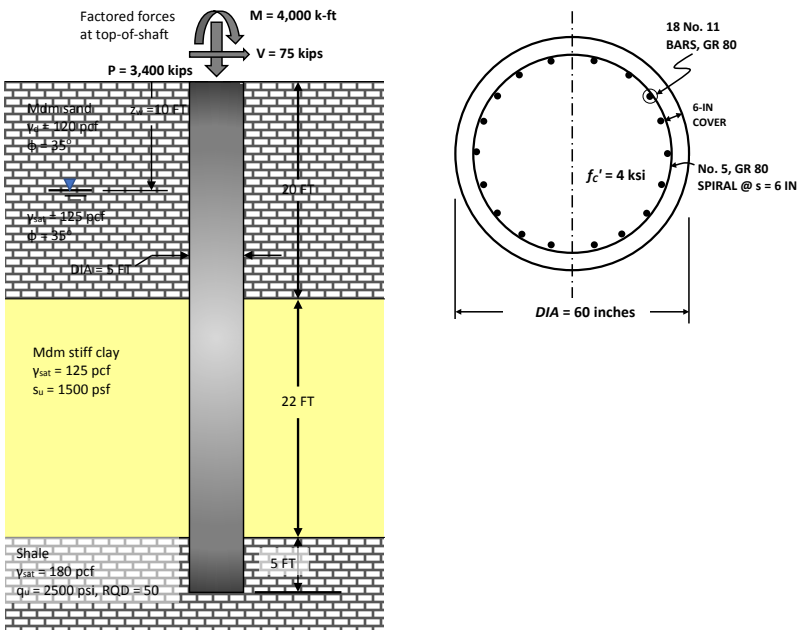
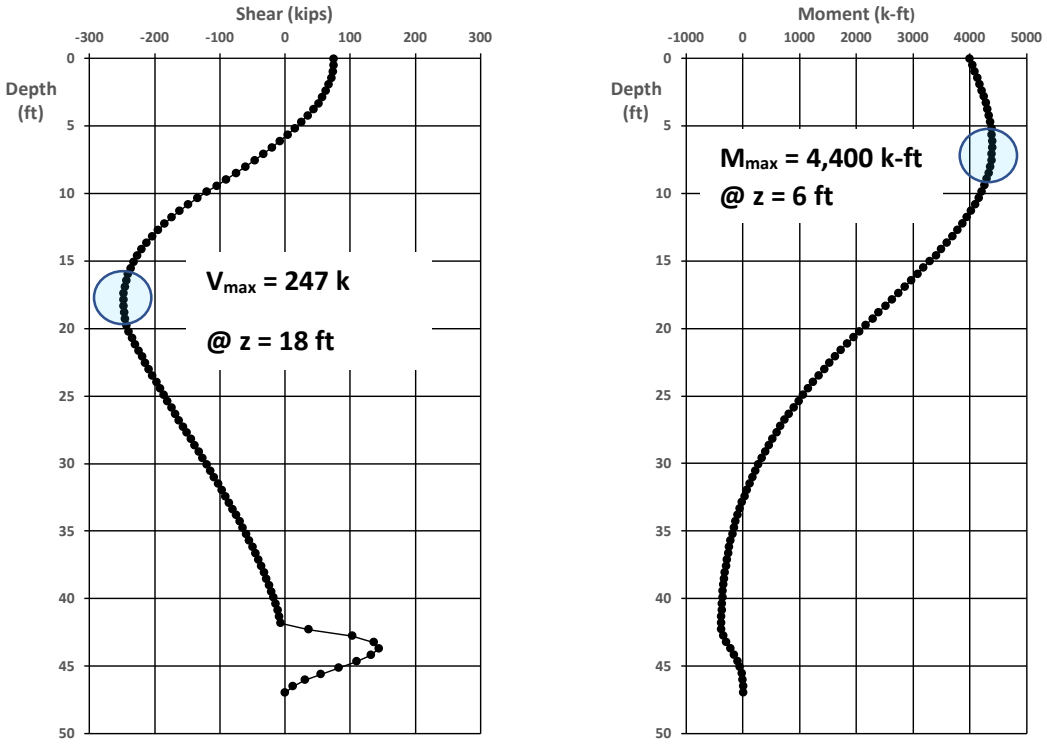


Figure 12-3 Example, trial drilled shaft for illustration of structural design. (a) subsurface profile and loads; (b) trial section



(a) (b)

Figure 12-4 Shear and Moment versus depth diagrams for example trial shaft shown in Figure 12-3.

12.7.4 Design for Combined Axial and Moment Loads

This step involves establishing the drilled shaft design parameters to satisfy the LRFD criterion that the factored combination of axial (P) and moment (M) demand does not exceed the factored combined resistance. The analytical tool used for this purpose is the P-M interaction diagram introduced in Section 12-6.

To evaluate whether a trial design satisfies the LRFD design criterion, the nominal P-M interaction diagram must first be factored by application of appropriate resistance factors, or:

$$P_r = \varphi P_n \qquad \text{(AASHTO 5.7.4.4-1)} \qquad 12\text{-}9$$

$$M_r = \varphi M_n \qquad \text{(AASHTO 5.7.3.2.1-1)} \qquad 12\text{-}10$$

where: P_r = Factored axial resistance
P_n = Nominal axial resistance
M_r = Factored moment resistance
M_n = Nominal moment resistance
φ = Resistance factor (see below)

The resistance factor (φ) varies as a function of the strain conditions governing the nominal strength of the section. The resistance factors are different for compression-controlled and tension-controlled sections. For strength load combinations falling into the compression-controlled range of the interaction diagram, the resistance factor is $\varphi = 0.75$. Compression-controlled is defined as a cross-section for which the net tensile strain (ε_t) in the extreme-fiber tensile steel at nominal strength is less than or equal to the compression controlled strain limit (ε_{cl}). Sections are considered tension controlled if the tensile strain (in the extreme-fiber tensile steel) at nominal strength is equal to or greater than the tension control limit (ε_{tl}). For strength load combinations falling into the tension-controlled range of the interaction diagram, the resistance factor is $\varphi = 0.90$. Figure 12-5 shows the variation of resistance factor as a function of the net tensile strain in the reinforcement. Values of the compression control strain limit (ε_{cl}) and tension control limit (ε_{tl}) for use in conjunction with Figure 12-5 depend on the grade of reinforcing steel as given in Table 12-2. For sections that transition between compression-controlled and tension-controlled, linear interpolation is used to determine φ as illustrated in Figure 12-5. The interpolation equation shown in the figure for nonprestressed reinforcement can be used to calculate transition-zone values of φ. Note that the same value of resistance factor is applied to the nominal values of P and M to generate the factored P-M resistances.

The LRFD criterion is met when each point representing a combined factored P-M demand lies within the factored interaction diagram (factored resistance). Figure 12-6 illustrates the concept. Points representing factored demand that fall in the zone labeled 'permissable' represent a structural design that satisfies the LRFD criterion.

When evaluating trial designs, assume a reasonable percentage of longitudinal reinforcement and generate the interaction (P-M) diagram. A typical starting point is $\rho_s = 1\%$, which is the minimum percent longitudinal steel. Check if the factored axial and moment demands fall within the acceptable zone on the interaction curve. If not, increase the amount and distribution of longitudinal reinforcement steel until the design criterion is met, subject to AASHTO design code limits. As noted in Chapter 6 (Rebar Cages) the designer has several options to meet the required steel percentage using multiple combinations of bar size,

grade of steel, spacing, and bundling of bars. Combinations that meets the structural demands while promoting constructability (for example by maximizing clear space between bars) should be selected for design.

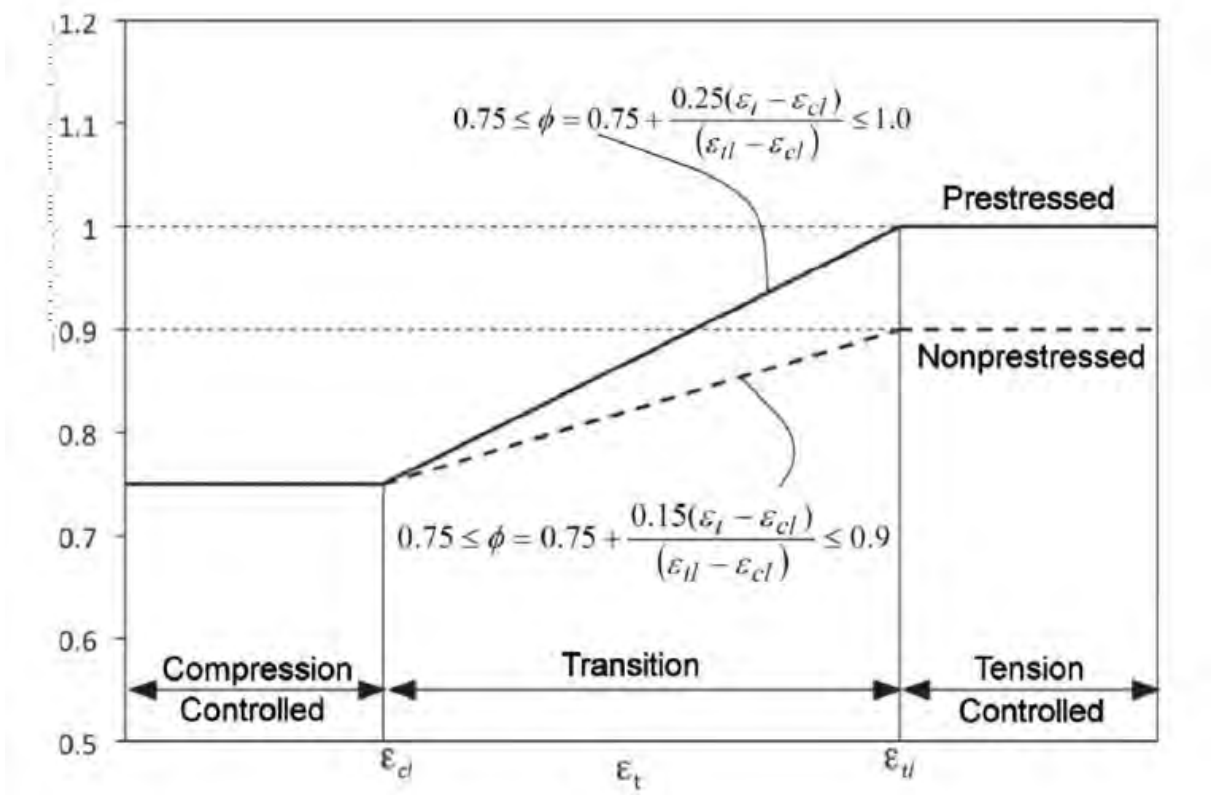


Figure 12-5 Variation of φ with Net Tensile Strain, ε_t . (AASHTO Figure C5.5.4.2-1)

TABLE 12-2 STRAIN LIMITS FOR NONPRESTRESSED REINFORCEMENT (AASHTO TABLE C5.6.2.1-1)

Specified Minimum Yield Strength, ksi	Strain Limits	
	Compression Control ε_{cl}	Tension Control ε_{tl}
60	0.0020	0.0050
75	0.0028	0.0050
80	00030	0.0056
100	0.0040	0.0080

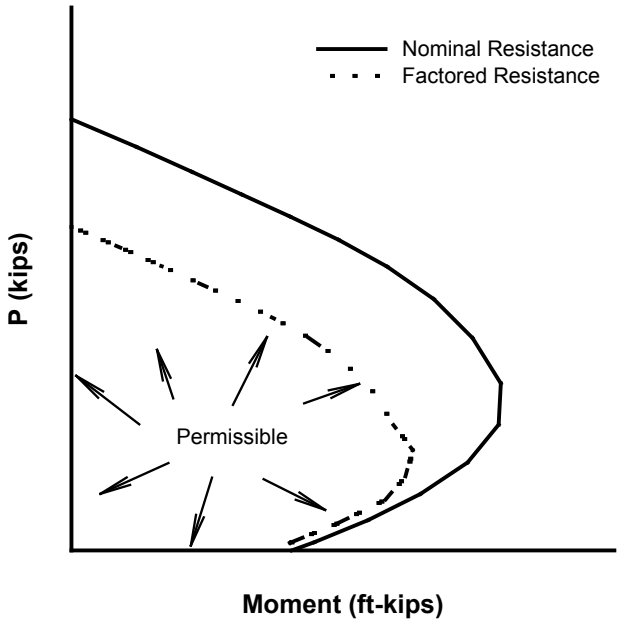


Figure 12-6 Nominal and Factored Interaction Diagrams

To illustrate the above design concepts for combined axial-flexural loading, consider the illustrative example and trial section shown in Figure 12-3. Based on analysis of the trial shaft under the factored loads at top-of-shaft, the factored moment diagram in Figure 12-4 shows that maximum moment demand is 4,400 k-ft and occurs at a depth of 6 ft. For the trial design section, consisting of eighteen Grade 80, No. 11 bars (f_y = 80 ksi), the nominal and factored P-M interaction diagrams are shown in Figure 12-7. The combined factored demand is plotted as a point on the interaction diagram with coordinates corresponding to P = 3,400 kips and M = 4,400 k-ft. The demand plots within the factored interaction curve, demonstrating that the trial section design satisfies the LRFD criterion. The steel ratio of approximately 1 percent provides clear spacing between longitudinal bars of 6.7 inches, which should be adequate to provide passing ability of a properly proportioned concrete mix design.

Cases involving combined axial tension and bending are analyzed by applying the same concepts described above for combined axial compression and bending. The portion of the interaction diagram where axial load is negative, as illustrated in Figure 12-7, would then govern the factored resistance. Also note that the strength limit state is always tension-controlled and therefore the resistance factor is φ = 0.90.

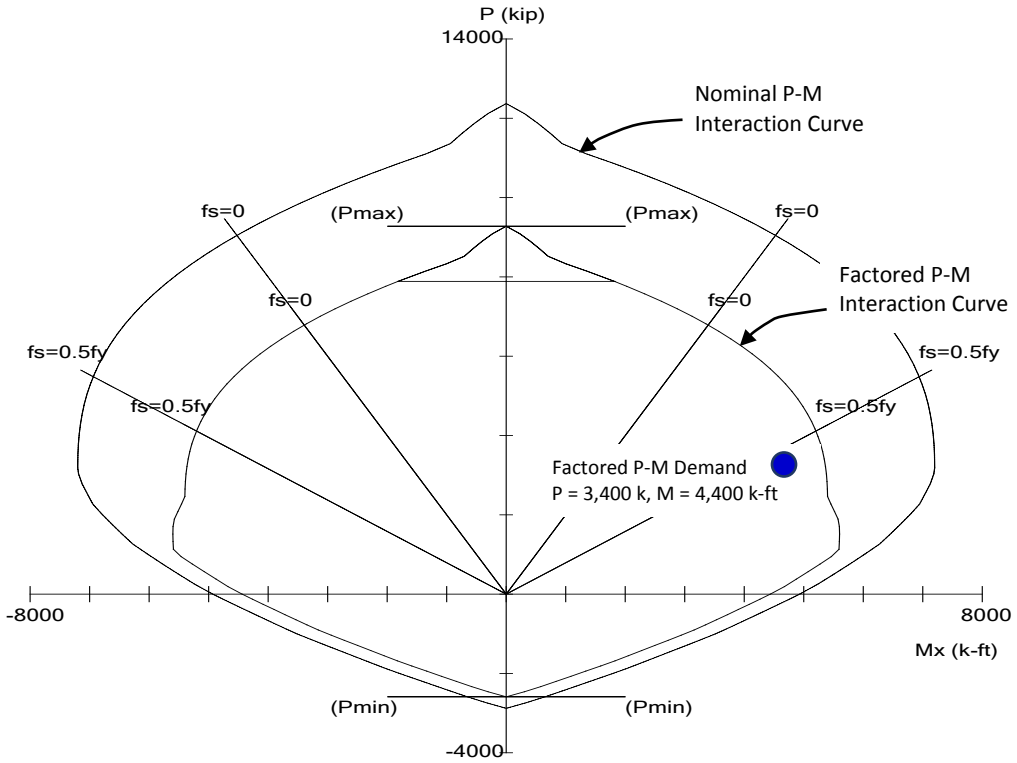


Figure 12-7 P-M Interaction diagram for illustrative example.

12.7.5 Design for Shear

As a first step, the designer should check whether the minimum amount of transverse steel required by the AASHTO design code, as given by Equation 12-5, is adequate to meet the factored shear demand for the section being considered. If so, the size and spacing of spiral or hoops are then selected to provide the minimum transverse steel. If not, the design is revised to increase the factored shear resistance. This can be accomplished in several ways including increasing the amount or grade of transverse steel or increasing the contribution to shear resistance provided by the concrete, by increasing the shaft diameter or increased concrete design strength.

The factored shear resistance V_r of a reinforced concrete section is calculated as:

$$V_r = \varphi V_n \qquad \text{(AASHTO 5.7.2.1-1)} \qquad 12\text{-}11$$

where: φ = resistance factor for shear = 0.90 for normal weight concrete
V_n = nominal shear resistance of the section,

The nominal shear resistance V_n is determined as the lesser of both of the following:

$$V_n = V_c + V_s \qquad \text{(AASHTO 5.7.3.3-1)} \qquad 12\text{-}12a$$

$$V_n = 0.25 f'_c b_v d_v \qquad \text{(AASHTO 5.7.3.3-2)} \qquad 12\text{-}13b$$

in which:

$$V_c = 0.0316\beta\lambda\sqrt{f_c'}b_v d_v \qquad \text{(AASHTO 5.7.7.7-3)} \qquad 12\text{-}14$$

$$V_s = \frac{A_v f_y d_v\,(\cot\theta + cot\,\alpha)\,sin\alpha}{s} \qquad \text{(AASHTO 5.7.7.7-4)} \qquad 12\text{-}15$$

where: V_c = shear resistance provided by the concrete
V_s = shear resistance provided by the transverse steel reinforcement
f_c' = design strength of concrete
b_v = effective shear width = section diameter (D) for circular members
d_v = effective shear depth = $0.9\left(D/2 + D_r/\pi\right)$ (see Figure 12-8)
D = external diameter of the drilled shaft
D_r = diameter of the circle passing through the centers of the longitudinal rebars
B = factor indicating ability of diagonally cracked concrete to transmit tension and shear
λ = concrete density modification factor
A_v = area of shear reinforcement within a distance *s*
f_y = yield strength of transverse steel
θ = angle of inclination of diagonal compressive stresses, taken as 45°
α = angle of inclination of transverse reinforcement to longitudinal axis
s = spacing of the ties along the axis of the member (spiral pitch)

Figure 12-8 shows a cross section of a circular reinforced concrete column (or drilled shaft) and defines the terms used above to describe its geometry. The portion of the cross sectional area effective in resisting shear is the product of the effective shear width and the effective shear depth, or b_v x d_v. Alternatively, the effective shear area may be estimated conservatively as 0.80 x A_{gross}.

Note that Equation 12-12 provides an upper-bound limit on the nominal shear resistance to ensure the concrete will not crush prior to yielding of the transverse reinforcement. While this should always be checked, in virtually all cases involving drilled shafts Equation 12-12 governs (*i.e.*, gives the lesser of the two values). Equation 12-12 takes into account the contributions provided by the concrete (V_c) and the transverse steel (V_s).

In Equation 12-13 for the concrete contribution V_c, the factor β can be taken as 2.0 for sections in compression as long as the section includes the minimum transverse reinforcement, and the factor λ is equal to 1.0 for normal weight concrete. This simplifies the expression for V_c to:

$$V_c = 0.0632\sqrt{f_c'}b_v d_v \qquad 12\text{-}16$$

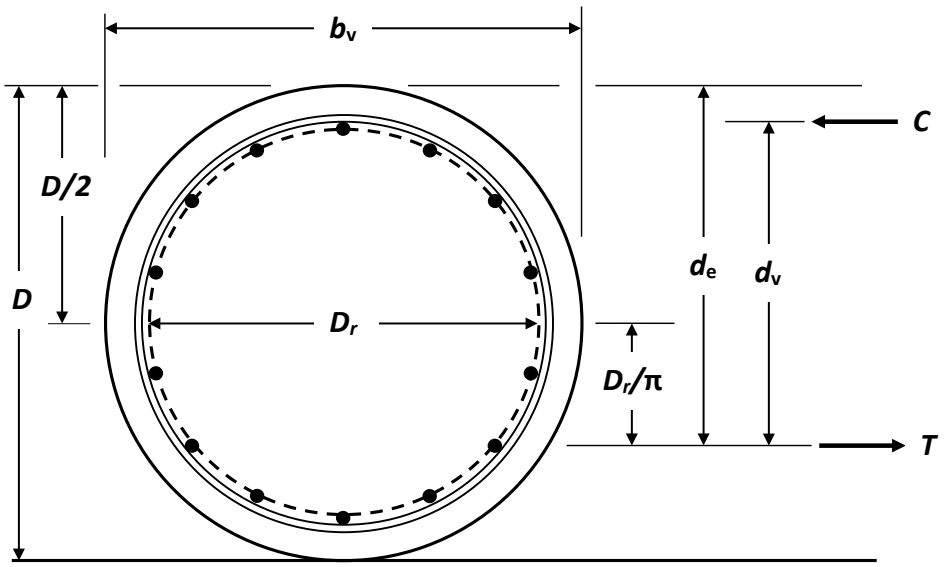


Figure 12-8 Illustration of terms b_v, and d_v for circular sections (after AASHTO Figure C5.7.2.8-2)

Considering Equation 12-14 for the steel contribution V_s, for the recommended value of $\theta = 45^o$ this expression reduces to:

$$V_s = \frac{A_v f_y d_v \ (\sin\alpha + \cos\alpha)}{s} \qquad 12\text{-}17$$

Equation 12-16 is applicable to spiral transverse reinforcement which is inclined at an angle α to the longitudinal bars. For hoops, $\alpha = 90^o$ and Equation 12-14 reduces to:

$$V_s = \frac{A_v f_y d_v}{s} \qquad 12\text{-}18$$

To illustrate the design for shear, consider the example introduced previously in Figure 12-3, with the shear and moment diagrams shown in Figure 12-4, and the interaction diagram analyzed as shown in Figure 12-7. The maximum factored shear demand V_{max} = 247 kips and occurs at a depth of 18 ft, as shown in Figure 12-4. As determined in Example 12-1, the code-specified minimum transverse steel can be satisfied by using Grade 80, No. 5 spiral on a 6-inch pitch. To calculate the nominal shear resistance provided by the No. 5 spiral with s = 6 inches:

For a 6-inch pitch and a spiral diameter of [60 - 12 + (5/8)] = 48.625 inch:

$\alpha = 90^o - \tan^{-1}[3"/48.625"] = 86.5^o$

d_v = effective shear depth $= 0.9\left(D/2 + D_r/\pi\right) = 0.9\ (60"/2 + 46.59"/\pi) = 40.4$ inches

b_v = effective shear width = D = 60 inches

Equation 12-15:

$$V_c = 0.0632\sqrt{f_c'}b_v d_v = 0.0632\ \sqrt{4\ ksi}\ 60\ in\ x\ 40.4\ in = \ 306\ kips$$

Equation 12-16:

$$V_s = \frac{A_v f_y d_v\ (\sin\alpha + \cos\alpha)}{s} = \frac{0.31\ in^2\ 80\ ksi\ \ 40.4\ in.(\sin 86.5 + \cos 86.5)}{6\ in} = 176.9 \text{ kips}$$

Equation 12-11:

$$V_r = \varphi\ V_n = 0.90\ [V_c + V_s] = 0.90\ [306 + 176.9] = \underline{\mathbf{435\ kips}}$$

Therefore, the factored shear demand of 247 kips does not exceed the factored shear resistance of 435 kips and the trial design with the minimum required amount of transverse steel satisfies the LRFD design criterion for shear.

As a check on the upper bound shear resistance, Equation 12-12b:

$$V_n = 0.25\ f'_c\ b_v\ d_v = 0.25\ (4\ \text{ksi})\ (60\ \text{in x}\ 40.4\ \text{in}) = 2{,}424\ \text{kips} > \ 482.9\ \text{kips}\ \ \underline{\mathbf{OK}}$$

Applying the AASHTO provisions described above in Section 12.3.2, transverse reinforcement is not required from a design perspective below a depth where the factored shear demand V_u decreases to a value less than one half of the factored shear resistance provided by the concrete (see Equation 12-4). Based on the calculations given above:

$$0.5\varphi V_u = 0.5\ (0.90)\ 306\text{kips} = 137.7\ \text{kips}$$

In Figure 12-4, the shear diagram for the example drilled shaft, the factored shear demand decreases to a value less than 137.7 kips at a depth of 29 ft. Below this depth, the transverse reinforcement vertical spacing could be increased, subject to the maximum spacing presented in Section 12.3.2, or more likely governed by considerations of safety and stability during cage handling as described in Chapter 6.

Furthermore, in Seismic Zones 3 and 4 the AASHTO design code states that transverse reinforcement for confinement shall be provided within drilled shafts in drilled shaft bents over a length extending to 3.0 times the diameter below the calculated point of moment fixity. The code does not provide a clear definition of moment fixity. A practical and conservative definition of moment fixity is the depth at which the moment demand is less than five percent of the maximum moment demand in the drilled shaft.

12.7.6 Review and Revise for Constructability

The final step of any drilled shaft structural design should consist of a constructability review. Questions that should be asked include:

- Is the clear spacing between longitudinal and transverse bars adequate to allow concrete flow?
- Is the design concrete cover adequate, allowing for tolerances on shaft location, rebar cage placement, and centering devices?
- Can the rebar cage be fabricated, handled, and lifted into place safely?

- Are there any design modifications that will make it easier to construct the drilled shaft?

If the answer to any of the first three questions is no, the design should be revised to promote constructability. If a drilled shaft cannot be constructed in a way that ensures concrete can be placed reliably with adequate cover of the reinforcing, performance of the drilled shaft is at risk, regardless of whether the code-specified design requirements are met. A successful design is one that meets the design objectives while also being constructible. The constructability review should be performed by personnel experienced in drilled shaft construction.

As stated in the Introduction, the objective of a final design, including the structural design, is to produce a set of plan drawings detailing the properties and geometry of the reinforced concrete shafts that will enable a qualified contractor to install the drilled shafts. Figure 12-9 shows an example of information provided in a set of plan drawings including the drilled shaft elevation view and cross sections. In this example a permanent casing extends to the top of rock and the shaft is uncased below the top of rock. The design shown on the drawings must be supported by calculations demonstrating that the LRFD criterion is satisfied for all applicable limit states and load cases, similar to those presented above to illustrate the steps involved in structural design.

12.8 ADDITIONAL STRUCTURAL DESIGN CONSIDERATIONS

12.8.1 Splices, Connections, and Cutoffs

A construction joint typically is located at the head of a drilled shaft, often close to the ground surface. This is also referred to as the drilled shaft cutoff elevation. If the drilled shaft is being used to support a column, the shaft may be of the same diameter as the column (*i.e*. Caltrans Type I shaft) or may be of a larger diameter than the column (*i.e*. Caltrans Type II shaft). In either case, if a joint is provided at the head of the drilled shaft, the drilled shaft and column reinforcement will require splices or connections. Construction aspects of reinforcement splices and connections are described in Section 6.6.

For a drilled shaft supporting a column, the column transverse reinforcement is required to be continued into the drilled shaft a distance of not less than one-half of the column or drilled shaft diameter, whichever is larger, or 15 inches minimum, in accordance with AASHTO 5.11.4.3. This requirement for extending the transverse reinforcement applies to all Seismic Zones and the objective is to avoid a plane of weakness at the interface created by the construction joint.

Furthermore, the nominal shear resistance at the construction joint shall satisfy:

$$V_n < 0.380\, b_v d_v \lambda \sqrt{f_c'} \qquad \text{(AASHTO 5.11.4.3-1)} \qquad 12\text{-}19$$

where: b_v = effective shear width = drilled shaft diameter
d_v = effective shear depth (see Figure 12-8)
λ = concrete density modification factor = 1 for normal weight concrete

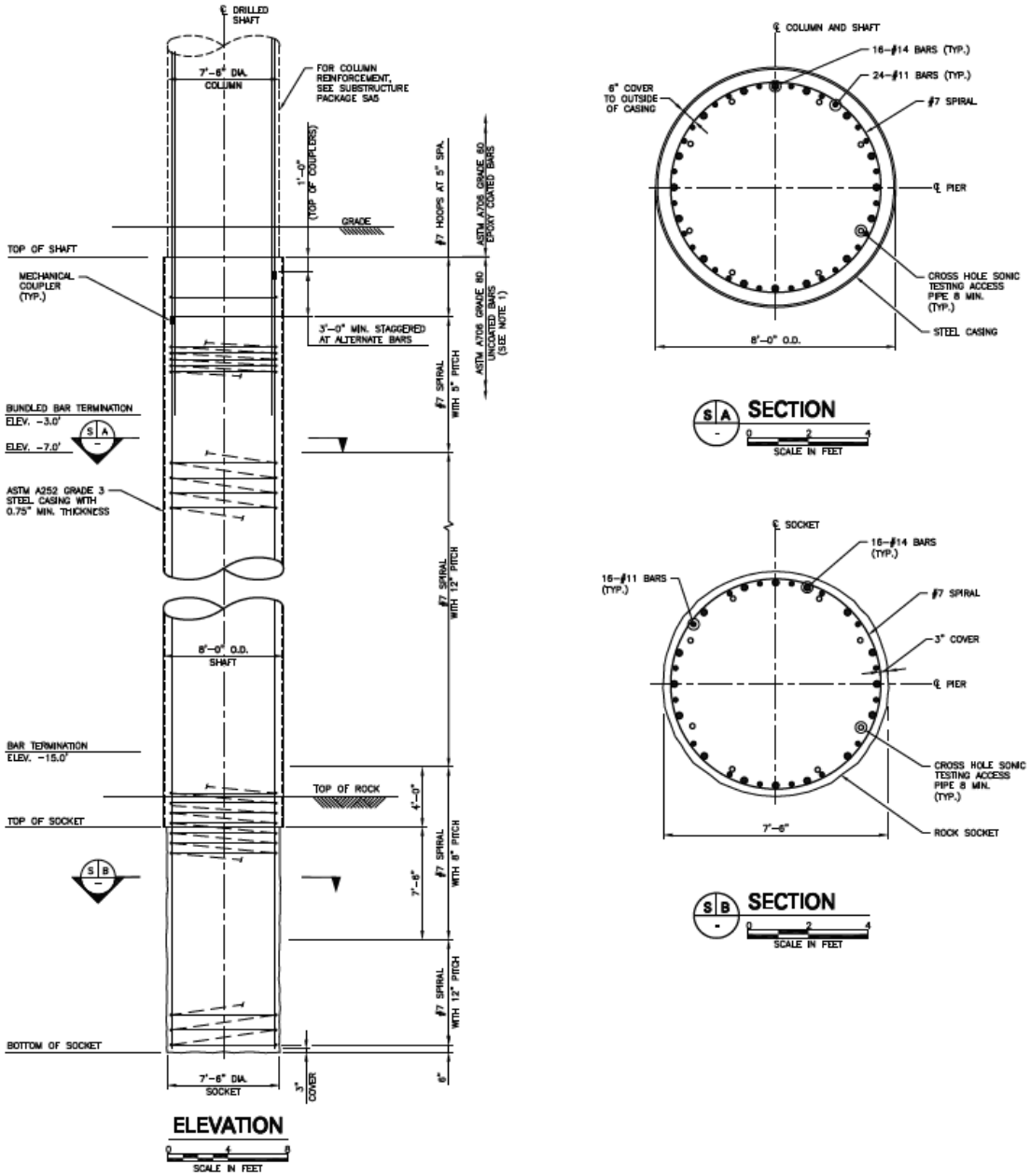


Figure 12-9 Example of elevation view and cross sections of drilled shaft from plan drawings.

For lap joints at the interface with the column, or for lap joints within the drilled shaft, the longitudinal shaft rebars should lap the longitudinal bars from the column or cap, or the bars in the adjoining section of cage, by the length required to develop the full yield strength of the reinforcement, l_d, as specified in Section 5.10.8 of the AASHTO design code. The same is true for the development of longitudinal drilled shaft reinforcement into a footing or pile cap. For Seismic Zones 2, 3 and 4, the lap length is increased by a factor of 1.25 to develop the over-strength resistance of the reinforcement, per AASHTO 5.11.4.3.

The current AASHTO design code, as well as ACI 318-14, prohibits lap splicing for No. 14 and No. 18 bars in tension and in compression, except that splices can be made between No. 14 and No. 18 bars with smaller bars for compression lap splices.

The length required to develop the full yield strength of the reinforcement, l_d, in inches, for bars in tension is taken as the product of the basic development length, l_{ab}, and the applicable modification factors listed under AASHTO 5.10.8.2.1b (modification factors which increase l_d), and AASHTO 5.10.8.2.1c (modification factors which decrease l_d). In no case should the development length l_{ab} be less than 12.0 inches for bars in tension.

For No. 11 bars and smaller, the basic development length for tension bars, l_{ab}, in inches, is taken as follows:

$$l_{db} = 2.4 d_b \frac{f_y}{\sqrt{f_c'}} \qquad \text{(AASHTO 5.10.8.2.1a-2)} \qquad 12\text{-}20$$

The current AASHTO design code does not provide development length equations for rebar sizes greater than No. 11. However, the modifications factors in AASHTO 5.10.8.2.1c incorporate the bar diameter, in addition to other factors including bar spacing and amount of transverse reinforcement, which allows the development length of No. 14 and No. 18 bars to be established. For most conditions, the following expressions provide a reasonable estimate:

For No. 14 bars:

$$l_{db} = \frac{2.70 f_y}{\sqrt{f_c'}} \qquad 12\text{-}21$$

For No. 18 bars:

$$l_{db} = \frac{3.5 f_y}{\sqrt{f_c'}} \qquad 12\text{-}22$$

The length required to develop the full yield strength of the reinforcement, l_d, in inches, for bars in compression is taken as the product of the basic development length, l_{ab}, and the applicable modification factors listed under AASHTO 5.10.8.2.2. In no case should the development length l_{ab} be less than 8.0 inches for bars in compression.

The basic development length for compression bars, l_{ab}, in inches, is taken as the greater of the following:

$$l_{db} \geq \frac{0.63 d_b f_y}{\sqrt{f_c'}} \qquad \text{(AASHTO 5.10.8.2.2a-2)} \qquad 12\text{-}23$$

and

$$l_{db} \geq 0.3 d_b f_y \qquad \text{(AASHTO 5.10.8.2.2a-3)} \qquad 12\text{-}24$$

In the above equations:

f_c' = the cylinder strength of the concrete at 28 days (ksi)

f_y = the nominal yield strength of the steel (ksi)

d_b = the diameter of the bar (inches)

A_{bs} = the cross-sectional area of the bar (in^2)

AASHTO 5.10.8.2.3 states that the development length of individual bars within a bundle, in tension or compression, shall be that for the individual bar, increased by 20 percent for a three-bar bundle and by 33 percent for a four-bar bundle. No increase is specified for two-bar bundles, which is the most common type used in drilled shafts.

It should be noted that, even though the drilled shaft as a whole may be in net compression for all loading cases, the longitudinal steel may still be in tension under some loading conditions due to bending effects. Generally, the development lengths for tension reinforcement should be used.

Similar rules apply to rebar cutoffs and to lapping of transverse steel.

For drilled shafts under axial tension only, as may occur under uplift loading, AASHTO Article 5.10.8.4.4 states that reinforcement splices shall be made only with full-welded splices or full-mechanical splices.

12.8.2 Drilled Shafts with Permanent Casing

As noted in Chapter 5 steel casing may be left in-place and becomes a permanent part of the drilled shaft foundation. This is always the case for drilled shafts extending through water and is sometimes the case for drilled shafts on land. Public owners sometimes stipulate the use of permanent casing to take advantage of its known benefits, which include:

- Casing provides longitudinal reinforcement on the outside perimeter of the foundation, which is the most efficient location for increasing flexural stiffness and resistance
- Provides continuous lateral confinement to the concrete which improves the strength and ductility of the concrete
- Casing prevents the concrete from spalling and provides a barrier between the concrete and soil (increased durability)

Figure 12-10 shows a completed drilled shaft with permanent casing, with the column reinforcing steel extending upward. Figure 12-9 shows an example of typical details for a drilled shaft constructed with permanent casing. In practice, structural design of a drilled shaft may or may not account for the contribution provided by the permanent casing. The following paragraphs discuss the current state-of-knowledge and design practice pertaining to the effects of casing on the structural behavior and structural design of drilled shafts.

Article 5.12.9.5 of the AASHTO design code states that “Shells that are more than 0.12 in. in thickness may be considered as part of the reinforcement.” This provision clearly allows the contribution of permanent steel casing to be taken into account for structural design. A minimum reduction in wall thickness of 0.06 in. is required to account for corrosion, but consideration should be given to increasing this reduction for casings that are directly exposed to salt water, especially in splash zones, or wherever sufficient data are available to make engineering estimates of corrosion loss. Section 5.2.3 of this manual provides additional discussion on corrosion losses for steel casings.



Figure 12-10 Completed drilled shaft with permanent casing.

Although AASHTO Article 5.12.9.5 allows the casing to be considered, there are currently no design provisions in Chapter 5 (Concrete Structures) of the AASHTO design code that provide specific guidance on how the casing is to be incorporated. Most commercial software used for analysis of deep foundations under lateral loading using the *p-y* method of analysis allows the user to specify permanent casing and performs a nonlinear moment-curvature analysis in which the steel casing is treated in the same manner as any other component of longitudinal reinforcing, *i.e.*, Bernoulli's assumption of 'plane section' is assumed to be valid. This analysis yields the nonlinear relationship between moment (M) and flexural stiffness (EI), which enables the designer to assess the increase in stiffness provided by the permanent casing. Some software packages also incorporate the permanent casing into the P-M interaction diagram, allowing the designer to evaluate trial designs with permanent casing using the same approach described in Section 12-7 of this chapter, Design Step 4 (Design for Combined Axial and Moment Loads).

For the assumption of 'plane section' to be valid, and therefore for the section analysis to be valid, the section must behave compositely, meaning there is no relative shear displacement at the interface between the inside of the casing and the concrete. NCHRP Research Report 872 (Bruneau et al., 2018) entitled "Contribution of Steel Casing to Single Shaft Foundation Structural Resistance" demonstrates that significant increases in strength occur whether or not composite behavior develops, *i.e.*, permanent casing provides strength benefits for both composite and non-composite behavior. This research also indicates that a minimum coefficient of friction of 0.5 between the steel casing and drilled shaft concrete is sufficient to develop composite behavior, and that this value matches the actual friction coefficient reported from experimental research. Even if composite behavior cannot be guaranteed, the strength of a non-composite section can be taken as the sum of the strengths of the reinforced concrete section plus that of the steel casing, substantially increasing axial and flexural strength compared to simply ignoring the contribution of the casing. Results and recommendations of the NCHRP report are not yet incorporated into AASHTO; at the present time it is up to the discretion of the designer or owner.

Also note that neglecting the effect of permanent casing on the stiffness of drilled shafts can lead to errors in the analysis of the superstructure, and can also lead to un-conservative estimates of foundation load demands. A structural model of a bridge in which foundation supports are treated as springs will incorporate the structural stiffness of the foundation elements. Under-estimating the stiffnesses by neglecting the casing will result in errors (over-estimates) in predicted deformations of the structure, and will often under-estimate the loads transmitted to the foundations. It is therefore recommended to always properly account for the contribution of permanent casing to drilled shaft stiffness, even if its contribution to strength is neglected.

As a design alternative to the type of moment-curvature analysis described above, a drilled shaft with permanent casing can be treated as a composite steel member, designated in Chapter 6 (Steel Structures) of the AASHTO design code as a 'concrete filled tube,' or CFT. Applicable sections of the code include Articles 6.9.2.2 (Combined Axial Compression and Flexure), 6.9.5.2.2 (Concrete Filled Tubes), and 6.12.2.3.2 (Flexural Resistance of Concrete Filled Tubes). Until further research provides more specific guidance, structural design based on either of the two approaches, *i.e*, moment-curvature analysis of the composite section, or methods applicable to CFT's, are suitable for structural design of drilled shafts with permanent casing.

12.8.3 Structural Design of Rock Sockets

Rock sockets pose some special challenges for the drilled shaft structural designer. An issue that is often encountered is the so-called 'shear spike' that appears in the results of a non-linear *p-y* analyses (Chapter 9). The magnitude of maximum shear occurring in the rock-socketed portion of the shaft is predicted to be significantly greater than the shear force transmitted at the top of the socket. This often leads to a design with high shear reinforcement, in some cases resulting in a highly congested cage posing problems for concrete passing ability in a part of the shaft where concrete integrity is critical.

A simple example of the moment and shear distribution for a rock-socked shaft is given in Figure 12-11. The example shows a 4-ft diameter drilled shaft socketed into rock. The analysis predicts the shear to be a constant 20 kips along the cantilevered portion of the shaft (top 15 ft) and then upon embedment into the rock, the shear changes sign and reaches a maximum value of -76 kips. The moment increases linearly and reaches a maximum of 300 k-ft at the rock surface, and then decreases with embedment. For comparison, shear and moment diagrams are also given for a cantilevered column fixed at the ground surface. One would expect the behavior of a cantilevered column to be similar to the behavior of a drilled shaft socketed into very strong rock (stronger and stiffer than the concrete of the drilled shaft). The shear and moment diagrams for the cantilevered portion of the drilled shaft and column agree exactly. However, the drilled shaft and the column differ considerably for the estimate of maximum shear. The maximum shear in the cantilevered column is 20 kips, while the maximum shear in the drilled shaft is predicted to be 76 kips. If the analysis for the drilled shaft is repeated with a stronger rock, the maximum shear becomes even greater.

This apparent magnification of shear is a direct result of the analysis method and the assumptions upon which it is based. The circular drilled shaft is treated like a conventional beam based on Bernoulli's plane section hypotheses, and its interaction with the surrounding rock mass is represented by discrete nonlinear springs (*p*-y curves) acting in one direction (horizontally). In reality, interaction between the concrete shaft and surrounding rock mass is more complex and is not captured adequately by *p*-y curves. As the drilled shaft rotates, the face of the shaft moves vertically (downward on the front face and upward on the back face) generating upward and downward acting shear stress over the cylindrical surface. Downward movement develops resistance in the rock which provides an additional component of rotational resistance to the drilled shaft. The drilled shaft is modeled as a beam that acts in bending only and ignores

the shear stiffness of the drilled shaft. As indicated in Figure 12-11, there can be significant discontinuities in the resistance compared to regions above the rock socket.

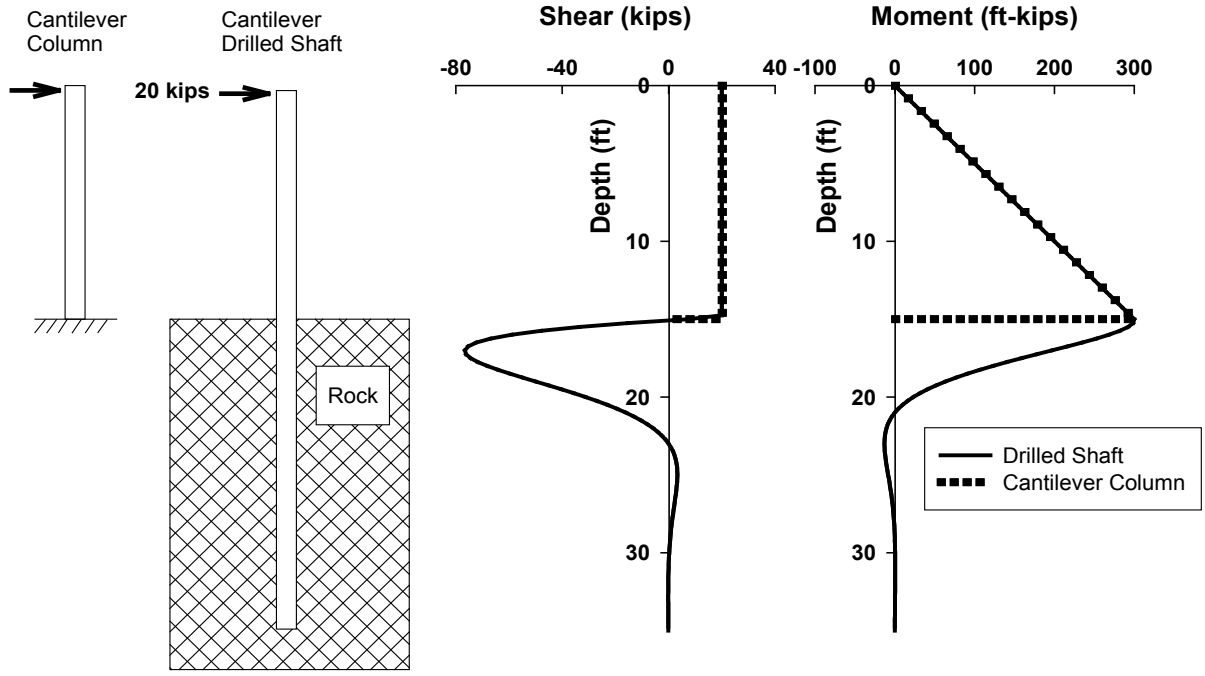


Figure 12-11 Comparison of Moment and Shear for a Cantilevered Column and a Docketed Drilled Shaft

Chapter 5 of the AASHTO design code provides two design methodologies for concrete structures. Each region of a structure is characterized as being either a B-Region (beam or Bernoulli) or a D-Region (disturbed or discontinuity). D-Regions occur in the vicinity of load or geometric discontinuities, where applied loads or support reactions cause complex variations in stress and strain, and therefore Bernoulli's hypothesis of plane section is not valid. Considering the complex interaction between rock-socketed drilled shaft and the surrounding rock, which acts more like a support than simply a 1-dimensional spring, rock sockets should be treated as D-Regions. For strength and extreme event limit states, the AASHTO design code specifies application of the strut-and-tie method (STM) for D-Regions.

In the STM, the D-Region of a concrete structure is modeled as an assembly of steel tension ties and concrete compressive struts interconnected at nodes to form a truss capable of carrying the applied loads to the supports. In the case of a rock socket, support is provided by the rock mass which completely encapsulates the cylindrical concrete socket. To apply the STM the rock support reactions must be resolved into discrete resultant forces located at nodes.

The steps required for a STM analysis are summarized as follows:

1. Define and isolate the section to be modeled
2. Calculate the resultant forces acting on the member section
3. Establish the truss geometry: axes of the struts and ties are chosen to approximately coincide with the axes of the compression and tension fields, respectively

4. Compute the forces in the struts and ties to transfer the resultant forces across the region; *i.e.*, the truss must satisfy external and internal equilibrium
5. Perform strength checks on each strut, tie, and nodal zone

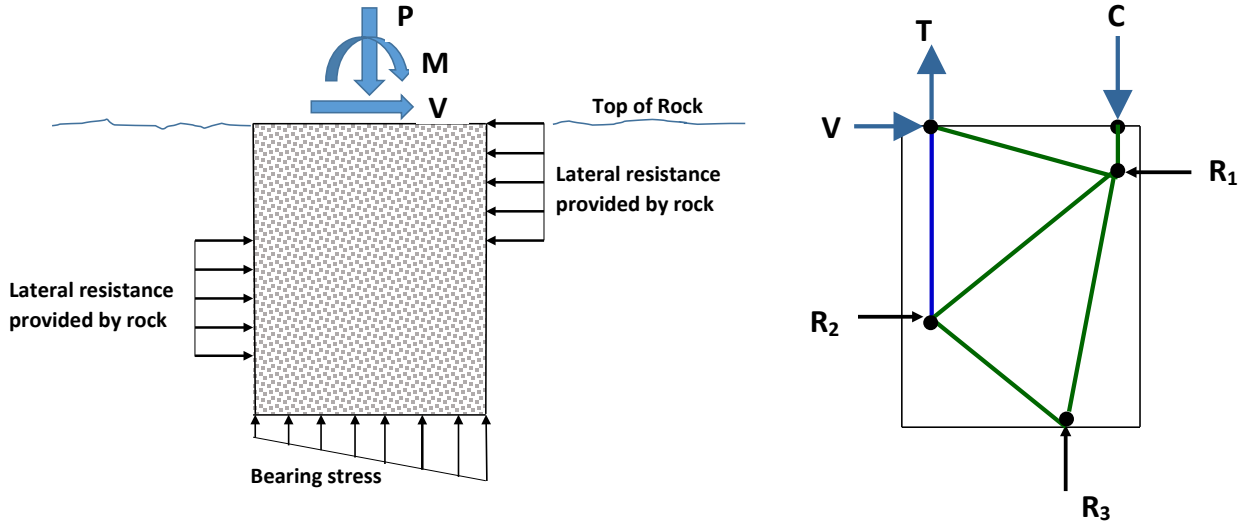
For a rock-socketed shaft, these steps can be further described as follows. Referring to Figure 12-12:

Step 1. The section to be isolated and modeled consists of the socketed portion of the drilled shaft measured from where the shaft enters rock to the socket tip elevation.

Step 2. Resultant forces acting on the isolated section (socket) consist of:

- Axial force (P), shear force (V), and moment (M) transmitted to the top of the socket; V and M determined using conventional B-region analysis, *e.g.*, p-y method (see Chapter 9)
- Resultants of the lateral resistance provided by the rock mass on each side of the socket (R_1 and R_2)
- Resultant of the bearing resistance acting at the tip of the socket (R_3)

Step 3. Truss geometry: Establish a node at the location of each support reaction provided by the rock (at R_1, R_2 and R_3). Replace moment (M) and axial load (P) at the top of the socket with an equivalent couple consisting of vertical tension (T) and compression (C) forces; establish nodes at the locations of T and C. Locate the axes of the vertical tie and vertical compression struts to coincide with locations of the equivalent tension (T) and compression (C_1) forces, respectively. Diagonal struts are selected such that strut end points coincide with nodes at R_1, R_2 and R_3. In Figure 12-12(b) the resulting truss formed by the single tie (in blue) and the five struts (in green) represents the internal structural resistance provided by the reinforced concrete structure.



(a) Actual loading / boundary conditions (b) Idealized truss for STM

Figure 12-12 STM for rock socket

Step 4. The force system is analyzed for equilibrium (statics) to compute the axial forces in the struts and tensile force in the tie (T).

Step 5. Strength checks are performed to determine: (a) if the longitudinal reinforcing provides sufficient factored resistance to the tension force demand of the tie (T), and (b) if the shaft concrete provides sufficient compressive resistance to the calculated strut forces.

12.9 SUMMARY

Structural design of drilled shafts, which are integral, load-bearing structural elements of a bridge or other structure, is covered in this chapter. Design methods follow the provisions in Chapter 5 (Concrete Structures) of the AASHTO LRFD Bridge Design Specifications, 8th edition (AASHTO, 2017a). A six-step process is outlined in which the reinforced concrete design of a drilled shaft is required to meet all factored load demands including axial compression and tension, shear, and bending. Code requirements for minimum longitudinal and transverse steel reinforcement are reviewed. Analysis methods used for design, including the application of P-M interaction diagrams, are presented with illustrative examples. The objective is to provide an overview with sufficient detail for a foundation engineer or structural engineer with basic understanding of reinforced concrete design principles to carry out the structural design of a drilled shaft. More detail can be found in Chapter 5 of the AASHTO design code. Some special considerations, including connections and splices, use of permanent casing, and application of strut-and-tie methods to structural design of rock sockets, are discussed. Design for constructability, an important theme of this manual, is emphasized throughout.

CHAPTER 13
FIELD LOAD TESTS

"One test result is worth one thousand expert opinions"
– Werner von Braun, rocket scientist, Huntsville, Alabama

13.1 GENERAL

In spite of the most thorough efforts to correlate drilled shaft performance to geomaterial properties, the behavior of drilled shafts is highly dependent upon the local geology and details of construction procedures. This makes it difficult to accurately predict strength and serviceability limits from standardized design methods such as those given in this manual. Site-specific field load tests performed under realistic and representative conditions offer the potential to improve accuracy of the predictions of performance and reliability of the constructed foundations. Because site-specific field load tests reduce some of the variability associated with predicting performance, the use of larger resistance factors are justified when load tests are performed at the project site.

Until recent years, field load tests on high capacity drilled shafts were quite rare due to the magnitude of the loads required to fully mobilize the resistance, as illustrated in Figure 13-1. Engineers and transportation agencies now enjoy the benefit of innovative test methods which allow cost-effective testing of foundations to loads and in ways never before possible. This chapter describes methods and interpretation of field load tests that can be used to improve the economy and reliability of drilled shaft foundations.



(a) (b)

Figure 13-1 Kentledge Static Load Tests Compared with (a) Bi-directional Load Test; (b) Force Pulse (Rapid) Load Test (photos courtesy Loadtest, Inc. and Applied Foundation Testing)

Load tests are performed for two general reasons:

- to obtain detailed information on load transfer in side and base resistance (or lateral soil resistance for a lateral load test) to allow for an improved design ("load transfer test"), or
- to prove that the test shaft, as constructed, can sustain a load of a given magnitude and thus verifying the strength and/or serviceability requirements of the design ("proof test").

A load transfer test is typically designed to try to fully mobilize the resistance of the soil or rock. The test often involves instrumentation of the test shaft to determine the distribution of side and base resistance (or lateral soil resistance for a lateral test). The data from such measurements can then be used to design or re-evaluate the design of the production shafts with more confidence than would otherwise be possible. Load transfer tests are most useful when performed on specific test shafts constructed in advance of production shaft installation. In this way, the load test results and lessons learned can be evaluated and incorporated into the design for optimum efficiency and reliability. Although the preferred approach is to perform load tests on non-production drilled shafts, in some instances the test shaft(s) may be incorporated into the production foundations. Where load tests are incorporated into production locations, there exists the risk of an unexpected low test result which would require mitigation of a production foundation.

Proof tests are typically designed to verify that the shafts as designed and constructed satisfy the strength and/or serviceability requirements of the project. Proof tests may be performed in advance of production or as a part of a verification program on actual production shafts. Proof tests can often provide benefits of reliability and may justify larger resistance factors, although at present there are no established procedures for performing proof tests nor AASHTO specifications addressing resistance factors for drilled shafts with proof tests. Unscheduled proof tests can be performed on production shafts when construction records indicate that the geotechnical resistance may be questionable.

13.1.1 Benefits and Limitations of Field Load Testing

When considering the possible use of field load testing on a drilled shaft project, it is useful to weigh the potential benefits and limitations of a field load test program. The relative benefits and limitations are related to the size and scope of the project, the potential difficulty of construction of the shafts, the schedule requirements, site access, the site geology, previous experience in the local area, and the sensitivity of the design to various geotechnical parameters. In weighing the potential benefits, it is generally prudent to perform sensitivity analyses of the foundation design during the design phase to evaluate the possible range of behaviors and the impact on costs and the schedule (Chapter 8).

Projects of significant size can benefit from field load tests in the following ways:

a. The test(s) provide a direct measure of resistance in the specific geologic formation in which the shafts will be founded.
b. The test(s) provide a direct measure of performance using the actual construction means and methods planned for the project.
c. The test(s) provide a means by which the design methodology can be refined in the local geologic conditions.
d. The overall reliability of the foundation is improved because of the site-specific verification of the design and construction methods.
e. Due to improved reliability, higher resistance factors can be used for design if field load testing is included in the project. This provides improved economy and efficiency even if no modifications to final shaft tip elevations are made after completion of testing.
f. The economy and efficiency of the foundations can be improved if refinements to the final design can be incorporated based on the test results.
g. Improved efficiency and possible reductions in shaft length or rock excavation can result in reduced risks of construction difficulties which could affect costs and/or the schedule.
h. The results can provide benefit on future projects.

While the benefits are significant, limitations of field load testing must be considered and include:

a. The results of measurements from a single or relatively few tests in a highly variable geology may be of limited benefit in evaluation of the possible variations in production shaft performance.
b. Field load tests require a substantial investment of resources and time. If the foundation construction is on the critical path of an accelerated schedule, the impact to the schedule from delays in starting production may be more costly than the additional shaft embedment resulting from a conservative design with lower resistance factors.
c. Projects with relatively few drilled shafts may not derive sufficient improvements in economy and efficiency to offset the costs of testing.
d. In some cases the design may not be sensitive to, or controlled by, geotechnical strength considerations. For instance, scour and lateral loading may dictate that the shafts be founded in a hard rock layer which has more than sufficient axial resistance even when assessed using a very conservative approach to design with an appropriately low resistance factor. In such a case, axial load tests may not represent a prudent investment of resources.

Often, the benefits can be enhanced and some of the limitations listed above can be overcome with careful planning and creative use of the load testing technologies described in this chapter.

13.1.2 Design-Phase Load Testing Program

The greatest benefit can almost always be derived from field load testing during the design phase of the project. Design-build (D-B) type contracts are often under the combined pressures of an accelerated schedule and incentives to achieve economical solutions. The D-B team may typically work together to execute field load tests while the design work is being completed so that the results can be used to achieve the maximum economy in the foundation design.

In conventional projects, a design-phase test program will generally require a separate contract with drawings, specifications, permits, etc. so that the work can be executed and the results implemented during final design. The benefits and limitations of this approach to field testing compared to performing the field load tests later in the project are summarized in TABLE 13-1.

13.1.3 Field Load Testing Program at the Start of Construction

Most field load tests are conducted during the construction phase of the project, after completion of final design but prior to start of production shaft installation. Although a design is completed at the time of the field load tests, the economic benefits of using the higher resistance factor associated with load testing can be used since the design is completed with the knowledge that field load tests will be performed. It is usually possible to incorporate some changes, such as adjustments to shaft tip elevation, that can provide additional benefit from the standpoint of economy and/or reliability. Note that an add/deduct price can be included in the contract for adjustments to length. The owner will not generally derive as much economy from reductions in shaft length as would be the case for a design-phase test program because the contractor's equipment must be sized for the maximum length and size of shaft that might be anticipated prior to the start of the project, and the contractor may have increased contingency in their bid.

The benefits and limitations of this approach to field testing compared to performing the field load tests during the design phase of the project are summarized in Table 13-2.

TABLE 13-1 SUMMARY OF BENEFITS AND LIMITATIONS OF DESIGN-PHASE FIELD LOAD TESTS

Benefits	Limitations
1. Results of testing can be readily implemented and design optimized for economy and constructability. 2. Comparative tests of alternative foundation systems can be performed, e.g. drilled shafts vs. driven piles. 3. Execution of field work in advance of bidding can reduce constructability issues for potential bidders. 4. Mitigation of constructability problems minimizes risk and therefore contingency costs in competitive bids and reduces risk of claims. 5. Completion of field testing in advance may allow construction to proceed immediately into production upon notice to proceed and thus may reduce construction time (assuming that similar construction methods are used for production shafts).	1. Time and effort is required to prepare contract documents for a separate load testing contract. 2. The time required to execute the testing program during design can be a problem if the project is on the "fast-track." 3. The costs of a separate mobilization and permits can be significant, especially if over-water work is required, or if there are nearby structures, traffic disruptions, or permit issues. 4. Some contractors may have little interest in bidding the load test contract if the magnitude of the contract is small, or if they feel that they have a competitive advantage in their construction techniques that they wish to keep confidential. 5. The performance of the test shafts may vary from the performance of production shafts if different methods of construction are used.

TABLE 13-2 SUMMARY OF BENEFITS AND LIMITATIONS OF FIELD LOAD TESTS AT THE START OF CONSTRUCTION

Benefits	Limitations
1. Design benefits from the use of higher resistance factors associated with load testing because of the assurance that field tests will be performed. 2. No separate mobilization or contract; requirements can readily be incorporated into the project specifications. 3. Avoids increased design time needed to procure, conduct, report and assess load tests. 4. Provides an opportunity to evaluate the specific construction methodology to be used on the project with respect to effect on performance. 5. Results can be incorporated into adjustments in shaft length.	1. Major changes to the design are not practical 2. There is no opportunity to test alternative foundation systems. 3. The financial benefits of reductions in length are less than would be realized from design-phase testing. 4. The time required for load testing on the front end of a project may impact the schedule, especially if the project is on an accelerated schedule. 5. There is often little time to evaluate the results and make changes because of potential delays in the start of production. 6. The contractor must bid on, size equipment for, and provide contingency for the least favorable conditions anticipated in advance of field load testing.

13.1.4 Proof Tests on Production Shafts

The preferred method of load testing is to perform load tests on drilled shafts constructed in advance of production installation; however, whether or not field load tests have been performed in advance of production shaft installation, proof tests on production shafts can provide quality assurance and improved reliability. Although the results cannot be readily implemented via design changes, the tests provide a verification of drilled shaft performance which includes the effects of the specific construction means and methods used on the project with minimal impact to the schedule. However, since an unfavorable test result on a foundation which has already been constructed can lead to inefficient mitigation work, designers understandably must include contingency resulting in a more conservative design when the verification testing is to come only on production shafts after construction has started. So, there is inevitably a hidden cost associated with this approach compared to load tests performed in advance of production shaft installation.

Some testing technologies may be limited or impractical due to potential adverse effects on the performance of production shafts compared to testing of a test shaft which is not incorporated into the foundation. For instance, a lateral test to large loads can produce non-reversible displacement and structural damage in flexure. A bi-directional load cell incorporated into a production shaft produces a discontinuity in the flexural reinforcement at the location of the cell. An uplift load which produces large upward displacement (more than about 1/2 inch) on a production shaft may have an adverse effect on the maximum side resistance which can be mobilized by a subsequent downward directed load. A test on a full size production shaft may be impractical due to the size and load required. These limitations do not preclude load tests on production shafts, but may limit the amount of useful information that can be obtained from such tests in some circumstances.

Proof tests may also be used in conjunction with pre-production tests to provide verification and quality assurance. In this way, proof tests can be used to evaluate variability in site conditions or construction methods. Shafts which have been constructed with some apparent deficiency can sometimes be evaluated with proof tests in lieu of expensive mitigation. Some methods of proof testing (described in more detail later in this chapter) are illustrated in Figure 13-2. In Figure 13-2(a), a dynamic load test was performed on a drilled shaft after the pier was constructed; in Figure 13-2(b) a rapid load test was performed directly on a production drilled shaft.

The benefits and limitations of proof testing compared to performing the field load tests prior to production shaft installation on the project are summarized in Table 13-3.

13.1.5 Field Load Testing for Research

Field load tests performed at representative locations within a geologic area can provide lasting benefits to a transportation agency through development of improved design methods and more reliable design parameters. Research-quality tests also afford an agency the opportunity to involve local universities and perhaps incorporate extensive instrumentation and measurements. In order that substantial benefit is derived from the investment in research quality field load tests, several challenges must be addressed, as follows:

a. The tests must be performed in a geological setting that is representative of conditions that are anticipated on future projects. Selection of an appropriate site may be the most challenging, but also the most important, aspect of the research.
b. The tests must be performed with construction procedures that are representative and typical for future projects.

c. Funding of field load tests from an agency's usually small research budget can be very difficult. If the research type tests can be incorporated into an upcoming construction project, then much of the costs of the field work can be born as a part of the construction work, and the research funds used to extend the benefit of the tests to future projects. This approach is an effective way for state and local transportation agencies to involve faculty and students from state universities in practical projects with research opportunities.



(a) (b)

Figure 13-2 Proof Tests on Production Shafts can Verify Axial Resistance

TABLE 13-3 SUMMARY OF BENEFITS AND LIMITATIONS OF PROOF TESTS ON PRODUCTION SHAFTS

Benefits	Limitations
1. May justify use of higher resistance factors in design 2. No separate mobilization or contract; requirements can readily be incorporated into the project specifications. 3. Provides an opportunity to evaluate the specific construction methodology to be used on the project with respect to effect on performance. 4. Little or no impact on the schedule. 5. A test on a questionable shaft can be performed to verify performance.	1. Test procedures and design resistance factors are not currently addressed in the AASHTO LRFD Specification 2. Changes to the design are not practical after production shafts are installed 3. The costs associated with an unfavorable test measurement necessitate a more conservative and thus more expensive design. 4. If result is unfavorable result, remediation of a production shaft may be required. 5. Possible effects of testing on production shaft performance must be considered. 6. Load tests on full size production shafts may be impractical due to size and loads required.

13.2 LOAD TESTS TO MEASURE AXIAL RESISTANCE

This section provides an overview of the most important considerations in planning field load tests to determine axial resistance of drilled shafts, along with a description of various methods for performing axial load tests, instrumentation for measurement of the performance of the shaft, and methods for interpreting the results of axial load tests.

13.2.1 General Considerations in Planning Axial Load Tests

Field load tests of drilled shaft foundations require a substantial investment of time and money, and therefore must be planned carefully to provide useful information. The overall objectives must be established and the details of the test program defined with appropriate consideration of the production shafts that the test is intended to model. A discussion of objectives and many of these important details follows.

13.2.1.1 Overall Objectives

The first step in planning axial load tests is to define the most important objectives of the testing program. Usually, the most important objectives relate to measurement or verification of the design parameters that are most critical to the foundation performance. These parameters should be identified during the design process (Chapter 8) by performing parametric studies of the sensitivity of the design to various resistance components if not intuitively obvious. For instance, if 95% of the axial resistance is derived from the rock socket, then the focus of the axial load testing will be upon the rock formation and its properties, while the character of the overburden is of little consideration. If the majority of the axial resistance is derived from end bearing on rock, then the important factors for the load test will be those relating to the character of the rock immediately below the test shaft compared to production shaft locations, and the cleanliness of the rock bearing surface; the embedded length into rock will be of secondary importance. If the drilled shaft derives most of the axial resistance through side shear, then the effect of the installation methods, drilling fluid, and exposure time may be primary considerations for the load test shaft. A plan for instrumenting the shaft may be very important in order to determine the distribution of load transfer in side shear so that the design can be refined.

A list of possible objectives from axial load tests is provided below. This list is by no means exhaustive, and additional project-specific objectives may be identified.

- Determine base resistance at a representative location in the bearing stratum
- Determine base resistance using a specific construction method and level of bottom-hole cleanliness
- Determine side resistance in a rock socket at a representative location in the bearing formation
- Determine side resistance with a specific construction method and drilling fluid
- Determine side resistance after the maximum allowed exposure time to drilling fluid
- Determine side resistance after the maximum allowed exposure time of an open hole in a rock which is prone to weathering and degradation
- Determine the benefits of sidewall grooving to side resistance (might include tests with and without grooving, for instance)
- Determine the distribution of side resistance in various strata, each of which may contribute to the total resistance

- Determine the side resistance at large axial displacement to verify that strain softening and brittle behavior does not occur
- Determine the contribution to side resistance of a portion of the shaft within permanent casing
- Determine the axial resistance below the scour zone by separating the portion of resistance above the design scour elevation
- Determine the effect of base grouting on base resistance in a representative location with specific grouting parameters (might include tests with and without grouting, for instance)
- Determine shaft load versus displacement relationships for both side and base resistance

In addition to the primary objectives related to axial resistance, secondary objectives might also be identified. Some secondary objectives might include issues relating to constructability of the drilled shafts, assessment of drilled shaft installation methods, effect of construction on nearby structures or vice versa, concrete mix performance, and others.

It is important that the primary objectives of the load test program be identified in the contract documents so that participants in the work who were not directly involved in the design are aware and contribute to the achievement of these objectives. These participants include the constructor, the inspectors, the testing specialists, the resident engineer, and possibly others.

13.2.1.2 Location and Number of Test Shafts

The current (2017) AASHTO design guidelines provide for increased resistance factors in design based on the number of load tests per site and a characterization of the variability of the site. A "site" may be defined as all or a portion of the area where a structure or structures are to be located. An understanding of the geology and stratigraphy of the project area is critical to determination of the appropriate number of load tests and/or delineation of more than one "site" for purposes of testing and interpretation of test results.

To relate the load test results to production shafts that are not load tested, it is necessary to match the static resistance prediction to the load test results, i.e. calibrating the static resistance prediction method. The calibration requires consideration of the geomaterial properties at the specific test location in accordance with the procedures outlined in Chapter 10 or with locally developed procedures for computing axial resistance. The calibrated static analysis method is then applied to other geologically similar locations across the site to determine the required shaft tip elevation for the load demands and specific geomaterial properties at the location of that shaft.

If the geologic or stratigraphic variation across the project area is so large that the extrapolation of a load test from the tested location to another production shaft location would be inappropriate, then multiple load tests should be considered. The interpretation of multiple load tests may be considered in two ways: 1) it may be appropriate to delineate the project area into more than one site for purposes of load testing and interpretation, or 2) it may be appropriate to consider the project area as a single site with multiple load tests intended to capture a range of material properties or conditions within an otherwise similar geologic setting. This determination requires judgment on the part of the design professional, as there is no simple definition of how much variability can or should be incorporated into a single site. Some general guidelines are offered below. Multiple load tests should be considered if any of the following are true:

- The geologic character of the predominant bearing formation is different; e.g., sandstone instead of shale, sand instead of clay, etc.
- The average calibrated resistance (unit load transfer in side shear or end bearing) in the zone providing the majority of the axial resistance varies from the test location by a factor of two or more,

- The location is so far from the test shaft location that the geologic or material character of the bearing materials may differ,
- The project includes a large number of drilled shafts; in general, it is recommended that a number of load test shafts should be at least 2% of the total production drilled shafts for the structure (not including miscellaneous lightly loaded foundation units).

Three examples are briefly described to illustrate the use of multiple load tests for projects spanning large areas or including large numbers of drilled shaft foundations. Each of these represent design-build projects in which the load tests serve the purpose of both validating the planned design methodology and the construction means and methods employed to execute the project.

Example number one is the Goethals Bridge Project between Elizabeth, NJ and Staten Island, NY. The project (Figure 13-3) included a cable-stayed bridge spanning about 2,000 ft over the Arthur Kill and long elevated approach structures on either side of the river, as well as an elevated crossing of the NJ Turnpike. The project and load testing program are described in more detail by Dapp et al. (2016) and Turner et al. (2016). Drilled shafts ranged in diameter from 4.5 ft to 10 ft, and although each of the pylons included a group of foundations, most of the roughly 200 drilled shafts were constructed to support columns under the approach piers. All of the drilled shaft foundations were designed and constructed as rock socket piers into the Passaic Formation, a sedimentary rock that is predominantly a siltstone with occasional interbedded shale layers. Axial resistance for design purposes was derived from both the weathered rock material and the underlying intact rock, with the former sampled using SPT methods and the latter using rock coring. Drilled shafts were constructed using permanent casing through the overlying soft alluvial sediments, which were considered for design only in terms of the lateral soil resistance.

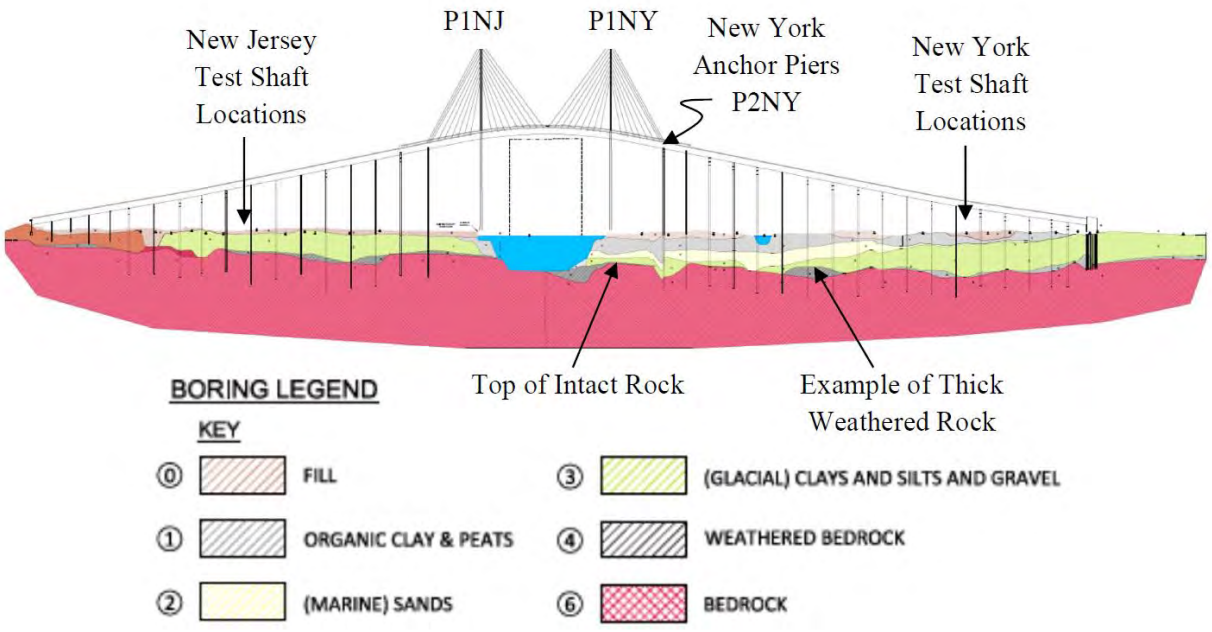


Figure 13-3 Schematic of Ground Conditions for the Goethals Bridge Project (Turner et al., 2016)

The load testing program for the Goethals Bridge included four axial load tests performed using embedded bi-directional load cells as described later in this chapter. Two tests were performed at a test site located near approach piers on both sides of the main span, with a test of a 5.5-ft diameter and a 9.5-ft diameter

rock socket diameter at each test site. The tests demonstrated the validity of the design methodology for a range of shaft diameters and conditions including both weathered and intact rock as well as the installation procedures used for construction. Lateral load tests were also performed on each test shaft after completion of the axial test. Although the project spanned a distance of nearly two miles, the geology was similar and the multiple load tests were considered to reflect conditions across one site comprising the entire alignment.

Example number two is the widening of a 1.5-mile portion of the elevated structure of the Lee Roy Selmon Expressway in Tampa, FL (Graham et al., 2013). The existing structure is founded on groups of driven prestressed concrete piles, and the widening required the addition of a column near the existing structure at most pier locations. A total of 237 additional 3.5 to 4-ft diameter drilled shafts were constructed, typically using a single drilled shaft to support a single additional column. The drilled shafts were founded in an area of the Tampa limestone which lies along the ancient Pamlico shoreline. The limestone in this formation is relatively soft, with compressive strengths typically in the range of 100 to 500 psi where the rock is sufficiently strong that cores can be retrieved. The more significant feature is the presence of highly weathered limestone with frequent solution cavities and soft soils which have infilled into solution features. The geology along this project can be characterized as quite consistent from a broader geologic perspective, but extremely variable on a localized scale from a foundation engineering perspective. Because of the extreme variability, the final design of each individual drilled shaft was developed from information obtained from a pilot hole boring located on the drilled shaft footprint.

The procedure used to calculate axial resistance was developed based on SPT testing in the pilot hole borings and confirmed based on five load tests across the site performed using the rapid load test methods described later in this chapter. The load tests (Figure 13-4) were constructed in a range of conditions reflecting various degrees of weathering within the limestone, and importantly evaluated a range of construction conditions which might be employed within the various conditions along the project alignment. For example, one technique used to stabilize the excavation through a solution feature is to backfill the excavation (through the cavity, which is often filled with extremely soft infill soils) using a low strength concrete and then re-drill the hole after this material has set. The installation plan was to use temporary casing to the top of rock and then complete the excavation using water only, but one load tested shaft was constructed using a bentonite drilling fluid to evaluate the effect, if any, on the measured side resistance relative to the values anticipated by the design. These construction variations were incorporated into the load testing plan and allowed flexibility with the installation methods that were implemented during construction of production foundations.

Example number three is a portion of the Honolulu elevated rail project in Hawaii. This project features an elevated rail structure, typically having 150-ft spans supported by single column piers with each column founded on a single 7 to 8-ft diameter drilled shaft. The first phase of the project to construct the guideway was awarded using a design-build contract, in which load testing was required as a performance verification of both the design methodology and construction techniques. This first phase project covered approximately 6-1/2 miles and crossed several very different types of geology. A total of nine load tests were performed to address both the geological variability and the associated differing construction methods planned along the alignment.

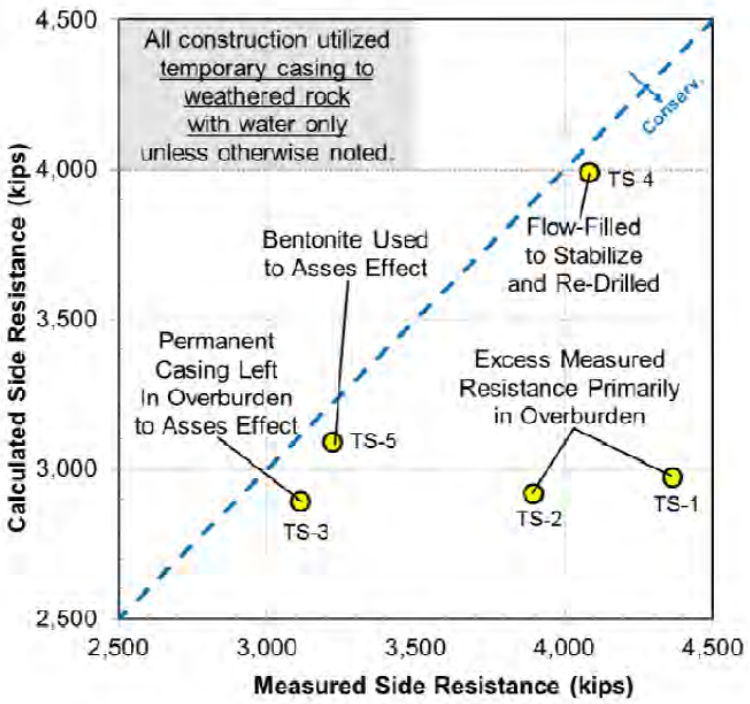


Figure 13-4 Summary of Load Test Results from the Selmon Widening Project (Graham et al., 2013)

Three load tests were performed in the westernmost 2-1/2 miles comprised of old alluvium. The soil conditions in this area were comprised of very stiff to hard clays that were overconsolidated by desiccation. Drilled shaft foundations in this portion of the project were typically less than 50 ft deep and were constructed in the dry using a crane-mounted auger drilling system.

Six load tests were performed over the remaining 4 miles of the project, with all these drilled shafts constructed in the wet. These six load tests provided measurements of side resistance in a wide range of soil materials including soft recent alluvium, stiff overconsolidated cohesive alluvium, granular overconsolidated alluvium, highly weathered basalt, and intact basalt bedrock. A schematic illustration of the ground conditions along a portion of the project is shown in Figure 13-5.

Three of these were founded with the base of the drilled shaft bearing in basalt bedrock, which had a variation in strength characteristics and with varying rock socket lengths. The drilled shafts constructed in basalt were installed using a temporary segmental casing to top of bedrock, which was installed and extracted using an oscillator attachment to the drill rig. The rock was then drilled using a rock auger.

The three remaining drilled shafts were constructed in alluvial soils to depths of up to 120 ft using water as the drilling fluid and with full length segmental casing. The base of each of these three drilled shafts was founded in a granular material composed of sand, silt, gravel, and cobbles, and then grouted to enhance the base resistance.

By addressing the variability of geology and construction methods broadly across the extent of this 6-1/2 mile project alignment, the program of load testing for the Phase I portion of the Honolulu Rail Project considered the verification of the design methods used to estimate axial resistance and the different construction methods outlined in the installation plan for the project.

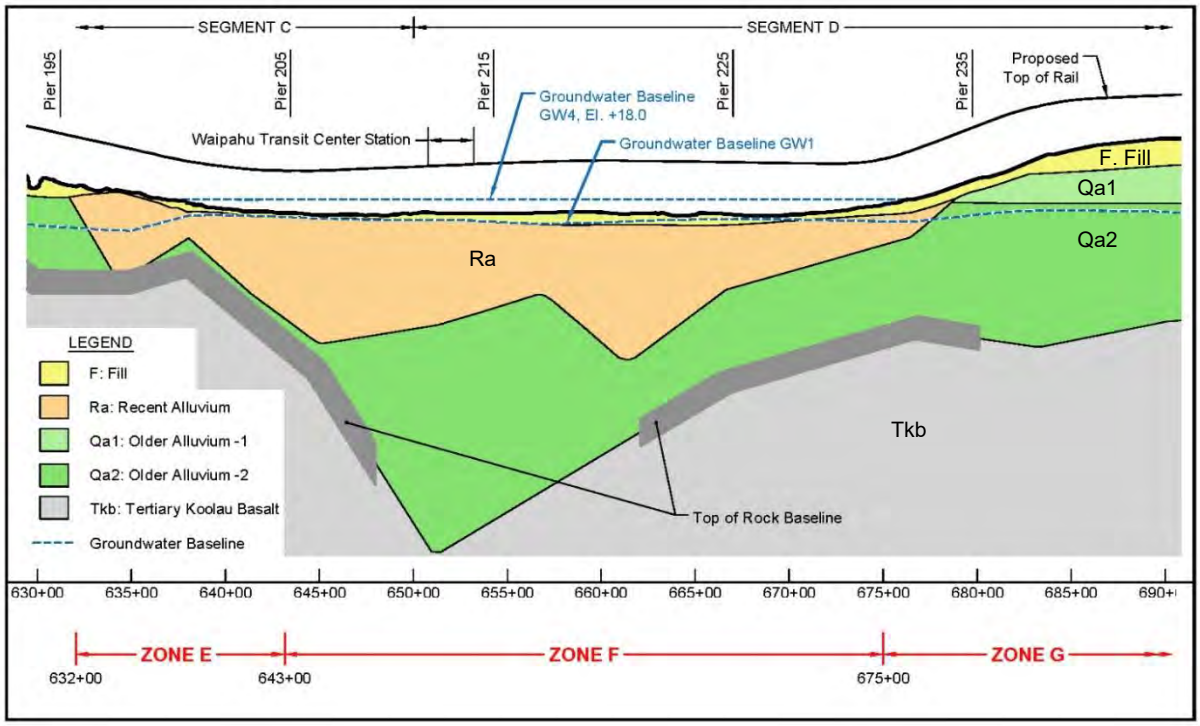


Figure 13-5 Schematic Illustration of Varying Ground Conditions for Honolulu Rail Project

13.2.1.3 Geo-Material Properties at Test Location

To appropriately interpret the results of an axial load test and calibrate them to the static resistance prediction method and design parameters, it is essential that the geomaterial properties at the specific test location be known with a high degree of confidence. The determination of geomaterial properties at the test location is accomplished using site investigation techniques such as borings or soundings, and from careful observation of the test shaft excavation.

A boring at the specific load test location is necessary, and the excavation of the test shaft should be carefully logged to verify consistency with the stratigraphy identified in the boring. A comprehensive program of in-situ and laboratory testing is required at the specific test shaft location to correlate axial resistance measurements with known material properties. Tests of representative rock core samples and detailed visual classification of the recovered rock cores are particularly important, since the strength of rock can exhibit significant variability.

Important geologic features may also be observable during excavation of the test shaft to a greater degree than is possible from a small diameter boring or rock core. Features such as boulders, irregular rock surface, cemented layers, and soft or weathered layers may be more readily distinguished during construction than during the site investigation, and these features may have an impact on the interpretation of the test results. In some weathered rock formations, it may be valuable to log the progress of the shaft excavation to correlate drilling rates with the degree of weathering. In this manner it may be possible to delineate zones of weak versus strong rock from the drilling activity, and more effectively correlate the load test results with performance of production shafts using inspection observations. Figure 13-6 shows material being removed from a rock coring bucket for visual examination.



Figure 13-6 Observation of Test Shaft Excavation Helps Define Geologic Conditions

13.2.1.4 Scour or Changes in Overburden Stress Conditions

There can be differences in stratigraphy and in overburden stresses between the test shaft and production shafts at other locations or for some design conditions due to grade changes or scour. In addition to the elimination of some layers which could contribute to axial resistance in the test shaft, changes in confining stress in the bearing zones can have a significant effect on the axial resistance. For these reasons, it is desirable to perform load tests on shafts at or near final grade, if possible. For cases with future scour, or where tests are performed in advance of mass grading, it may not be possible to load test the shaft under conditions similar to those of the final production shaft, and some accommodation is required. It is often possible to separate axial resistance from scourable overburden from the test shaft by using an isolation casing or by determining the resistance in these zones from strain gauge instrumentation. The effects of changes in stress within the bearing strata on the axial resistance of the shaft must also be rationally assessed and included in the interpretation of the results, as discussed in Section 13.2.4.

13.2.1.5 Construction of Test Shaft

Since construction techniques can have a significant effect on the measured axial resistance, the test shaft must be constructed in a manner consistent with that to be used on production shafts. For this reason, it is advisable that a test installation (often referred to as a "technique shaft" or "demonstration shaft") be performed prior to installation of the load test shaft so that any questions relating to final installation methods can be resolved in advance. Some factors that are considered to be important include:

- Drilling fluids. As discussed in Chapter 5, there can be differences in side resistance for different drilling fluids (bentonite, polymer, or water) in some soil or rock formations.
- Use of casing. The use of casing, especially casing which is advanced ahead of the shaft excavation, can have an effect on sidewall roughness and the lateral stresses around the shaft, as discussed in Chapter 5. The type of casing, method of advancement, and the presence of cutting teeth on the leading edge of the casing can influence the axial resistance.
- Drilling tools. An auger or digging bucket may result in different sidewall roughness than a coring barrel, and such differences can affect axial resistance. The test shaft excavation should be accomplished using similar tools to those used on production shafts.
- Bottom conditions. The tools used to excavate and clean the shaft base should be similar to those used for production shafts. The degree of cleaning at the base should be consistent with, but not superior to, those used for production in order that the measured axial resistance is consistent with production shafts. The use of inspection tools such as downhole cameras may be used to verify cleanliness and, in some cases, "calibrate" more routine inspection methods such as sounding.
- Time the excavation is open. Since some soil or rock formations can weather or decompose in the presence of air or fluid after the shaft excavation is complete, the test shaft program should be designed to capture any such effects which are unavoidable on production shafts. For instance, if the installation plan or the specification for production shafts anticipates up to 2 days to excavate, clean the base, place the reinforcement, and place the concrete after starting the excavation of a rock socket in shale, then the test shaft should be constructed under similar circumstances.

The inspection and observation of the test shaft excavation is an important component of documenting the geologic features and ground conditions at the specific test shaft location, as discussed in Section 13.2.1.3.

The "as-built" dimensions of the test shaft are important for interpretation of the test results, particularly if any significant deviations from the planned dimensions are observed. The actual shaft dimensions also affect the calibration and interpretation of strain measurements. Concrete volume measurements should be routinely obtained as a part of the inspection process, as described in Chapter 15. Careful measurements of concrete volume placed as a function of height in the shaft provide a means to determine the average area of the cross section over the shaft length between measurements. However, the as-built conditions obtained from concrete volume measurements are not very precise, and a borehole caliper provides a more effective means to determine the actual test shaft dimensions. Borehole caliper measurements are recommended for test shafts to better define the irregularity on the shaft cross section and to assist in the evaluation of the load test data.

Borehole calipers can be either the mechanical type with outreaching arms to "feel" the side of the hole, or a sonic caliper which remotely senses the borehole wall from a suspended probe. Examples are illustrated in Figure 13-7.

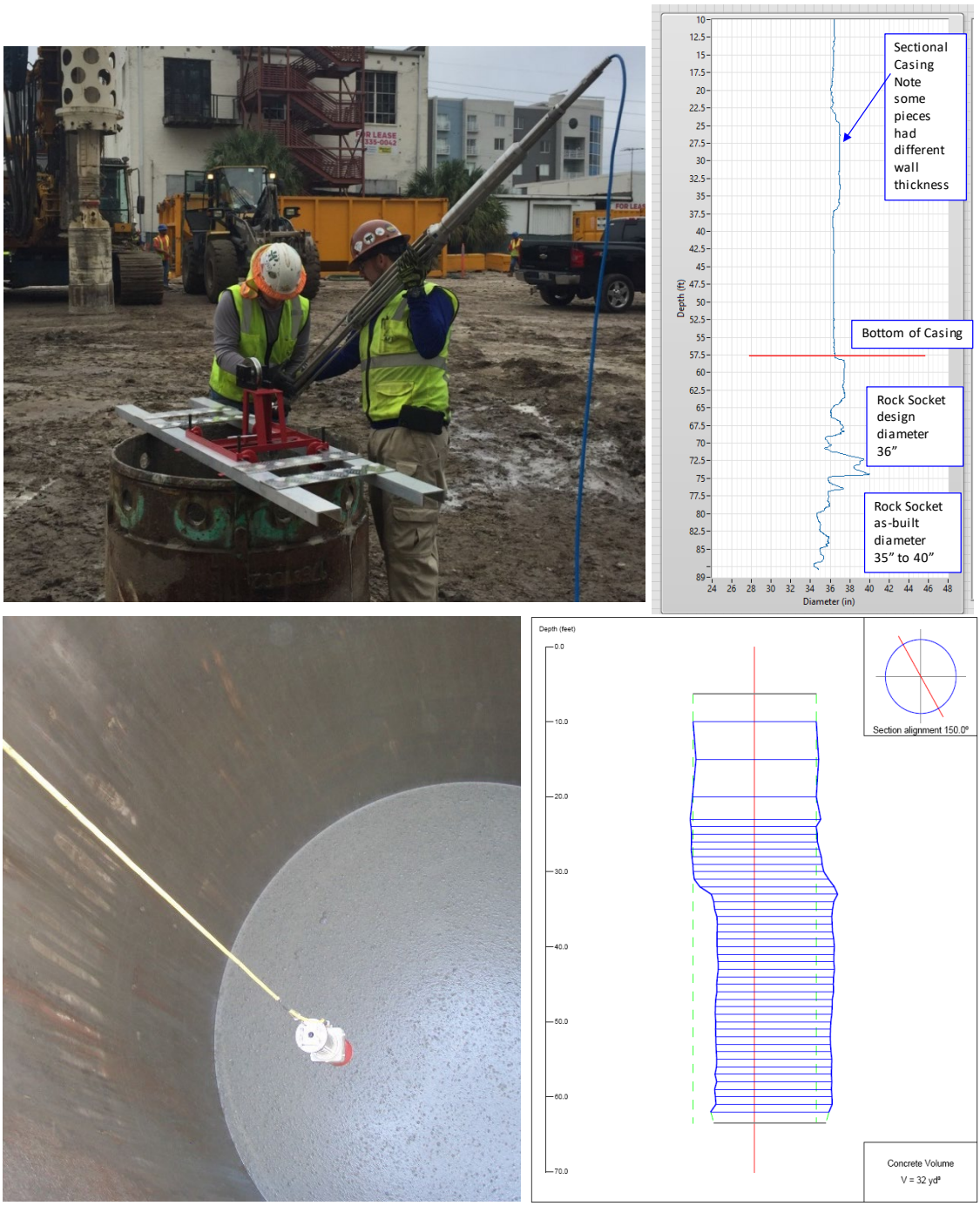


Figure 13-7 Example Borehole Calipers & Logs; Mechanical (top, photo courtesy Applied Foundation Testing) and Sonic (bottom, graphic image courtesy Loadtest, Inc.)

13.2.1.6 Use of Prototype Shafts

The magnitude of loads required to fully mobilize the resistance of large, high capacity drilled shaft foundations can make tests on production size shafts impractical in some instances. If prototype shafts are to be considered for use as load test shafts, some general guidelines are suggested below.

- The test shaft should be at least 1/2 the diameter of the production shaft, and should not be less than 30 inches diameter.
- The test shaft should be constructed using similar tools and construction techniques, as described above in Section 13.2.1.5. When the exposure time of an open or slurry filled hole is important, the test should replicate the exposure time of the full-size production shaft.
- The displacements at which unit values of side and base resistance are mobilized should be interpreted with respect to diameter as described in Chapter 10 in order to extrapolate to the anticipated performance of production size shafts.

Even if the guidelines above are followed, the use of prototype test shafts introduces additional uncertainties regarding the direct measure of the performance of a full-size production shaft. Load tests should be conducted on production sized shafts whenever possible or practical. Where prototype load tests are performed on test shafts that are significantly smaller than full production sized drilled shafts, the designers should consider that the use of the higher resistance factors associated with full size load tests may not be justified.

Verification load tests (that serve a primary purpose of justifying the higher resistance factors associated with site specific load testing) should be performed on full production sized drilled shafts that are located in representative ground conditions and constructed in a manner similar to the production drilled shafts. In this way, the load tests capture the various aspects of the excavation tooling, drilling fluids or casing, effects if any of the time of construction, and other variables described in this manual in a manner that is generally representative of production conditions. Where a range of production drilled shaft sizes are planned, load tests should generally target the drilled shafts that represent the more challenging conditions represented by diameter and depths of production drilled shafts. Some engineering judgment is always required to balance costs and benefits and the need to capture the range of conditions anticipated, and the previously cited case histories provide examples for consideration.

The remainder of this section provides some information on the current state of the practice relative to potential size effects on axial resistance, some of which are inherently incorporated into current design methods.

Besides the different displacement required to mobilize unit base resistance related to shaft diameter, the zone of influence below the base of a smaller diameter test shaft is less than a larger production shaft. Therefore, the relative influence of stratigraphy below the base could be a factor affecting maximum unit end bearing between shafts of different diameter.

There is evidence that very small diameter shafts (such as micropiles) may have significantly greater side shearing resistance per unit area compared to the much larger sizes typical of drilled shafts (Lizzi, 1983; O'Neill et al., 1996). The effect of diameter on unit side resistance is considered to be more significant in rock than in soil.

The effect of diameter on mobilized resistance is thought to be largely related to the effect of sidewall roughness and dilatancy, and the effect of dilatancy on the radial stresses generated as the shaft resistance is mobilized. Dilatancy would be expected to produce an increase in normal stress at the shaft/rock interface which is greater for a smaller diameter shaft compared to a larger one having a similar magnitude of sidewall

roughness. However, for large diameter shafts this effect is expected to be small. Presented on Figure 13-8 are the results from an analytical study of this effect by Baycan as reported by Miller (2003). This figure suggests that one would expect little difference in maximum unit side shear between a 4.9-ft (1500 mm) diameter rock socket and a 6.6-ft (2000 mm) diameter rock socket.

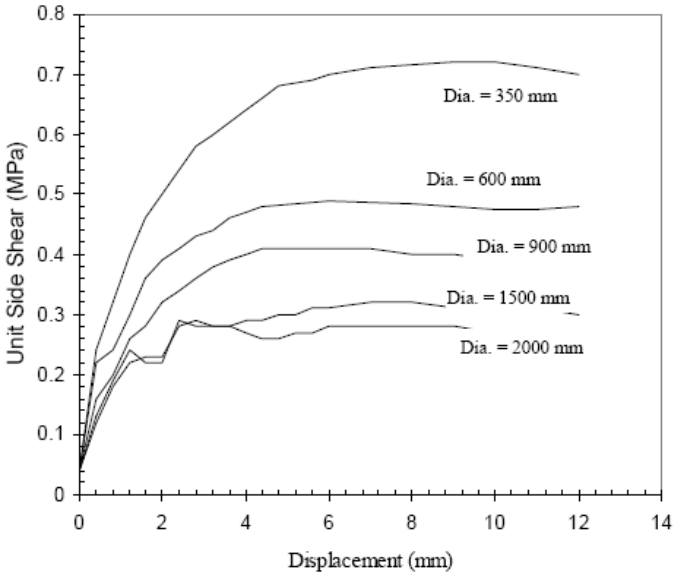


Figure 13-8 Unit Side Shear vs Displacement for Drilled Shaft Sockets in Rock of Moderate Roughness with q_u = 450 psi (Miller, 2003; Baycan, 1996); 1 inch = 25.4 mm, 1 MPa ≈ 147 psi

Observations from load testing programs including drilled shafts of various sizes suggest that the measured unit side resistance is relatively independent of diameter for drilled shafts constructed using similar excavation techniques. Castelli (2004) described the results of load tests on drilled shafts ranging in diameter from 5 ft to 8 ft in the Cooper Marl Formations in Charleston, SC and found similar side resistance values in the upper, non-cemented marl where multiple sized drilled shafts were tested. Similar results were obtained for drilled shafts ranging from 3 to 5 ft diameter at the Fuller Warren Bridge in Jacksonville, FL (Castelli and Fan, 2002). Load tests of the 5.5 and 9.5-ft diameter rock socketed drilled shafts at the previously described Goethals Bridge near New York (Turner et al., 2016) showed no differences in axial resistance relative to diameter to the maximum test loads, although the nominal side and base resistance was not fully mobilized.

The simple methods outlined in Chapter 10 for design of drilled shafts for axial loads do not specifically account for the effect of diameter on axial resistance in side shear. However, some axial computation methods suggest that there could be an inverse power relationship between shaft diameter and maximum unit side shear for "rough" rock sockets. For example, Horvath et al. (1983) proposed a relationship for artificially roughened sockets in rock as indicated in Equation 13-1:

$$f_{\max} = 0.8\left[\frac{\Delta r}{r}\left(\frac{L'}{L}\right)\right]^{0.45} q_u \qquad 13\text{-}1$$

where: f_{max} = maximum unit side shear
q_u = compressive strength of rock
Δr = height of asperities or grooves in sidewall
r = radius of shaft

L' = distance along surface of socket
L = length of socket

For a given magnitude of sidewall roughness, the relationship of Equation 13-1 would suggest that the maximum unit side shear in a prototype shaft relative to a larger production shaft would be as illustrated on Figure 13-9. The data from Baycan cited above are also plotted relative to the 6.28-ft (2,000-mm) diameter socket, and indicate excellent agreement for shafts in the 1 to 6-ft diameter range with the trend predicted using the Horvath equation.

The data and relationship plotted in Figure 13-9 were developed for shafts in rock which have significant roughness and would be expected to exhibit significant dilatancy at the shaft/rock interface. The data are limited to a maximum shaft diameter of 6.3 ft. The trend for smoother sockets or for shafts in soil is likely to exhibit a less significant effect of diameter, and the trend shown might reasonably be considered as an upper bound.

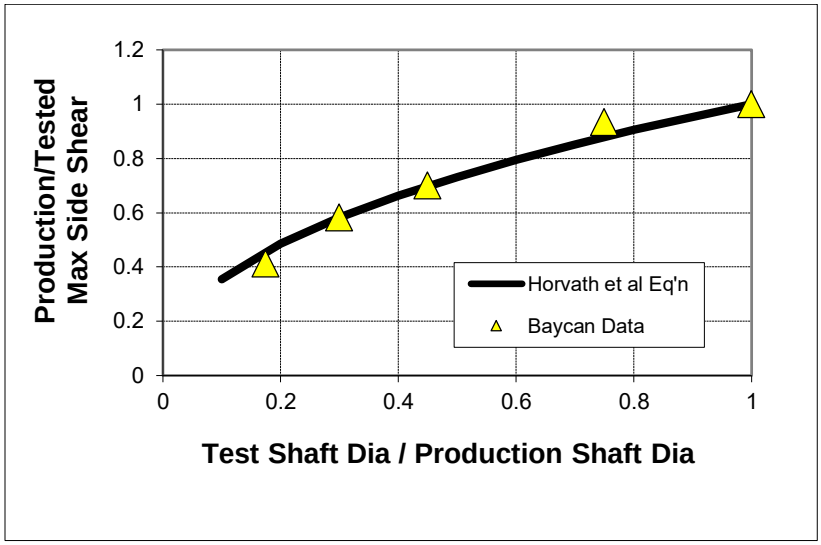


Figure 13-9 Computed Relationship Between Shaft Diameter and Maximum Unit Side Shear Resistance for Rock Sockets using Horvath Equation (13-1)

The magnitude of load required to perform a top-down test on a high capacity shaft can be many thousands of tons. Bi-directional testing (described in Section 13.2.2.2) provides a means of testing high capacity shafts by using an embedded jack to engage the side resistance as a reaction against the base resistance. However, if the base resistance greatly exceeds the available side shear, then it may be impossible to load a production size shaft and fully develop the base resistance during the test. Since the area available for side resistance is proportional to the shaft diameter, while the area available for base resistance is proportional to the *square* of the shaft diameter, it may be possible to balance the anticipated base and side resistance in a test shaft by using a smaller diameter.

Another possible solution to testing a drilled shaft with large base resistance is to utilize bi-directional testing with a production size shaft in side shear as a reaction against a smaller base area. This approach might be particularly effective in testing a shaft extending through soil overburden to engage end bearing on rock. Commonly called the "Chicago method," a schematic diagram of this type of test is illustrated in Figure 13-10 for a test of a rock socket with limited embedment. For the test shaft shown in that figure, the bi-directional cell acts by pushing the 36-inch diameter base area against the side shear of a socket which is 48 inches diameter. Additional details of this approach are described in Section 13.2.2.2.

The interpretation of a test performed as illustrated in Figure 13-10 would require that the load versus displacement relationship for the smaller prototype base be adjusted for the diameter of the production shaft to combine with the side shear and compute the overall load versus displacement response of the production shaft. More information on the interpretation of load test data is provided in Section 13.2.4.

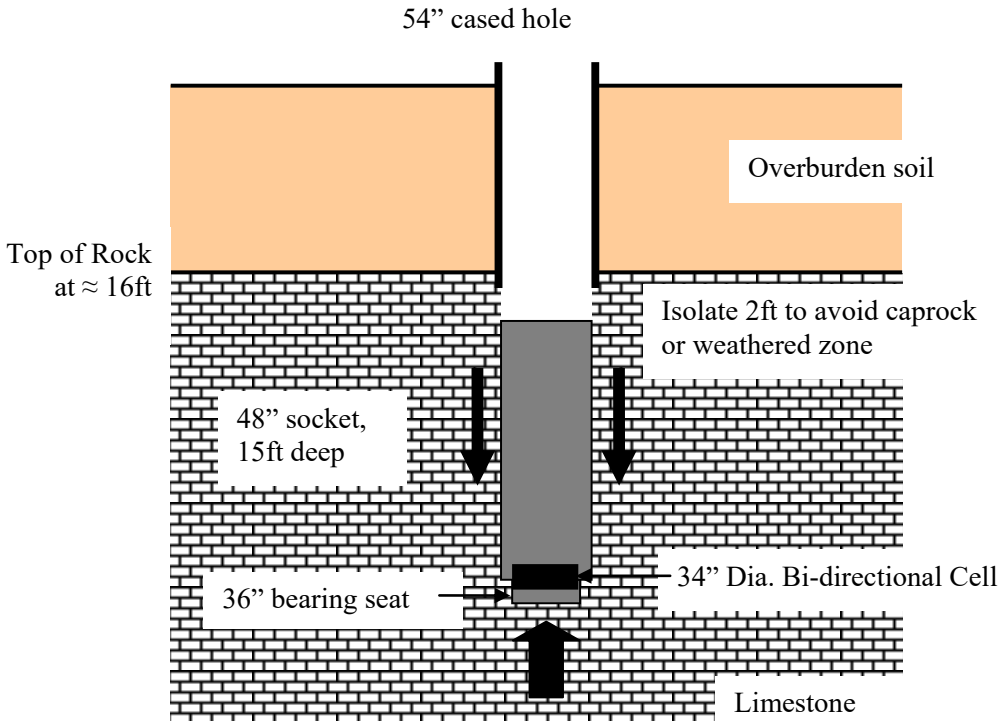


Figure 13-10 “Chicago Method” Load Test using Bi-directional Cell

13.2.1.7 Group Considerations

If the production shafts are to be installed in groups, it should be recognized that their behavior may be different from that of an isolated single load test shaft. Although tests on groups of relatively small diameter shafts might be performed as a part of research testing to evaluate group effects, tests on groups of drilled shafts are rare due to the magnitude of load required and the costs to perform such tests. Adjustments to the anticipated load-displacement behavior of shafts in a group from that of a single shaft should be made following the general guidelines and procedures outlined in Chapter 11.

13.2.2 Test Methods

The predominant methods used for load testing drilled shafts include conventional top-down static load tests with a hydraulic jack and reaction system, bi-directional testing using an embedded jack, top-down rapid load testing (using either (a) combustion gas pressure, or (b) a cushioned drop mass), and high strain dynamic load testing using a drop weight impact. Each of these methods has advantages and limitations in certain circumstances and experienced foundation engineers (like mechanics) know how to use all the tools in their toolbox. A brief description of each of these methods is provided below.

13.2.2.1 Conventional Top-Down Load Test

The most reliable method to measure the axial performance of a constructed drilled shaft is to apply static load downward onto the top of the drilled shaft in the same manner that the shaft will receive load from the structure. Static load testing of deep foundations has a long history and is described in FHWA-SA-91-042 (Kyfor et al, 1992). Tests should generally be performed in accordance with ASTM D1143. For small (3 to 4-ft) diameter shafts in soil, conventional static load testing to loads of up to 1,200 tons can be performed in a reasonably economical manner. Conventional static load tests have been performed on occasion to loads of up to 4,000 tons. However, the high load capacity and large diameter of many drilled shafts limit the ability to perform conventional static load tests because of the load requirements imposed on the reaction system.

13.2.2.1.1 Reaction System

The most common reaction system used with a conventional static load test is comprised of a reaction beam with an anchorage system, as shown in Figure 13-11. This photo is of a 1,200 ton capacity load test. The key components of the reaction system that affect the load capacity of the test include the reaction beam, the anchorage, and the hydraulic jack used to apply the load. Considering the very large loads and stresses in the reaction beam and anchorage, the entire reaction system should be designed by a professional engineer who is experienced with load testing.

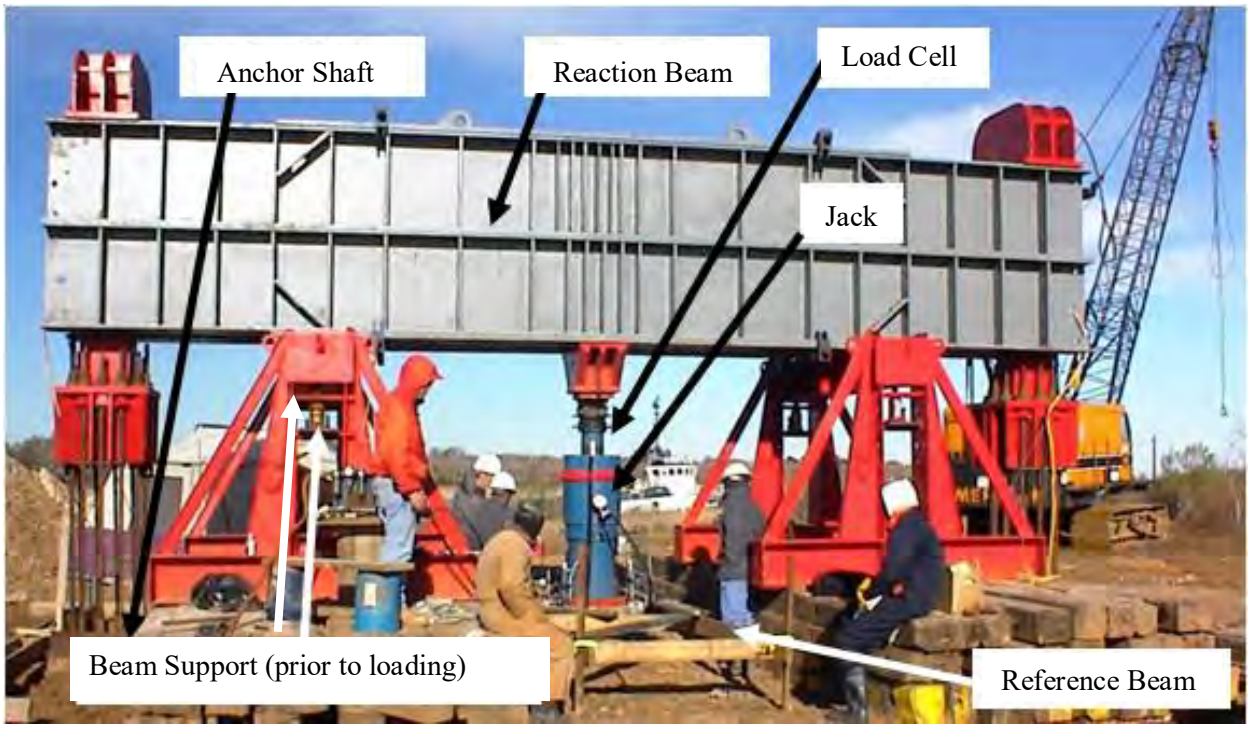


Figure 13-11 Conventional Static Load Test on a Drilled Shaft (photo courtesy Bill Isenhower)

The anchor foundations can consist of drilled shafts (as shown in Figure 13-11), with threaded rods used for longitudinal reinforcing), micropiles, grouted anchors, or driven piles such as steel H piles that can be extracted and salvaged. In some cases, such as the "Kentledge" tests shown in Figure 13-1, dead weight may be used as a reaction to avoid the need for anchor foundations.

Anchor foundations must be constructed so that the installation and use of these foundations do not affect the performance of the drilled shaft to be tested. Installation of driven or vibrated piles below the test shaft base can have an adverse effect on the base resistance by fracturing the bearing stratum or by causing ground subsidence of granular soils below the base. If grouted anchors are installed below the base of the test shaft, the stresses from these anchors can be superimposed on the bearing formation in the vicinity of the test shaft and affect the base resistance.

ASTM D-1143 requires that a clear distance of at least five times the maximum of the diameter of the test pile or anchor pile must be maintained between the test pile and anchor piles. This requirement can be impractical to achieve for large diameter drilled shaft foundations. A spacing of 3.5 diameters is recommended as a more feasible alternative, provided that the adverse affects of reaction pile installation cited above can be avoided.

The location of the anchor foundations affects the length of the beam, and the bending stresses in the reaction beam increase dramatically with the span length. For large shafts and large loads, a system of four or more reaction shafts with a multiple beam reaction frame may be used, such as the 4,000-ton capacity system employed by Caltrans and illustrated on Figure 13-12. And just to illustrate the extremes of testing, the photo in Figure 13-13 is the largest static load test of a drilled shaft known to the authors, a 5,700-ton test performed in Taiwan.

The reaction system must also be designed to avoid twist or eccentric loading in the reaction beam(s) during the activation of the hydraulic jack(s). This requirement is much easier to accomplish if the reaction anchors are designed to allow horizontal adjustment of the beam so that it can be centered and leveled over the test shaft. If only two reaction shafts are used and the reaction beam positioned directly over the reaction shafts, the beam may not line up precisely over the test shaft. If the jack is not precisely perpendicular to and centered on the reaction beam, then the beam will tend to twist and apply eccentric forces to the jack, load cell, and any bearing plates within the system. Eccentric forces in the jack and load cell can damage the equipment and produce inaccurate and unconservative measurements. In addition, eccentricity in the loading system is very dangerous and can cause plates or other pieces to be ejected explosively due to the stored strain energy in the system.



Figure 13-12 4,000-ton Capacity Reaction System (photo courtesy of Caltrans)



Figure 13-13 5,700-ton Capacity Load Test in Taiwan (photo courtesy Prof. San-Shyan "Sonny" Lin)

13.2.2.1.2 Loading Procedures

The recommended loading procedure for static testing follows the ASTM D1143 "Procedure A: Quick Test" loading method. This procedure requires that the load be applied in increments of 5% of the "anticipated failure load" which should be interpreted as the nominal axial resistance of the shaft. Each load increment is maintained for at least 4 minutes, but not more than 15 minutes, using the same time interval for all increments. After completion of the test, the load should be removed in 5 to 10 equal decrements, with similar unloading time intervals. Other loading procedures are optional, including maintained loading, constant rate of penetration, and others.

Load, displacement, strain, and any other measurements should be recorded at periods of 0.5, 1, 2 and 4 minutes, and at 8 and 15 minutes if longer intervals are used. Periodic measurements of the movements of the reaction system are also recommended in order to detect any unusual movements which might indicate pending failure of the anchor system or other component. A discussion of measurements and instrumentation is provided in Section 13.2.3.

13.2.2.2 Bi-Directional Load Testing

The bi-directional load test is performed by applying the load with an expendable jack (or multiple jacks) sandwiched between an upper and a lower load plate cast within the test shaft. The test is performed by using the upper portion of the drilled shaft as a reaction against the base and lower portion of the shaft, as illustrated in Figure 13-14. Some of the first known bi-directional tests were reported by Gibson and Devenney (1972) who used a hydraulic jack placed at the bottom of a borehole in rock, and by Horvath and

Kenney (1979) who performed tests using a stacked series of mining-type flat jacks embedded in a rock socket. A more effective jacking system was developed in the 1980's by Professor Jorj Osterberg (1992, 1994), leading to widespread use of this system for drilled shaft load testing. An ASTM standard has now been adopted (ASTM D8169-18) for load testing using the bi-directional loading method.

The principle and operation of a bi-directional test is simple. After a drilled shaft has been cast with the embedded bi-directional cell assembly, and the concrete has had sufficient time to gain sufficient strength for testing, the cell(s) is pressurized to break the tack welds holding the cell(s) together and to "crack" the shaft into an upper and a lower portion. The pressure may then be reapplied incrementally to begin imposing the bi-directional load to the upper and lower shaft sections. During testing, the top shaft section provides reaction for the bottom section, and visa versa. Because the shaft must be separated at the location of the bi-directional load cell, there cannot be continuous reinforcement across the location of the load cell.

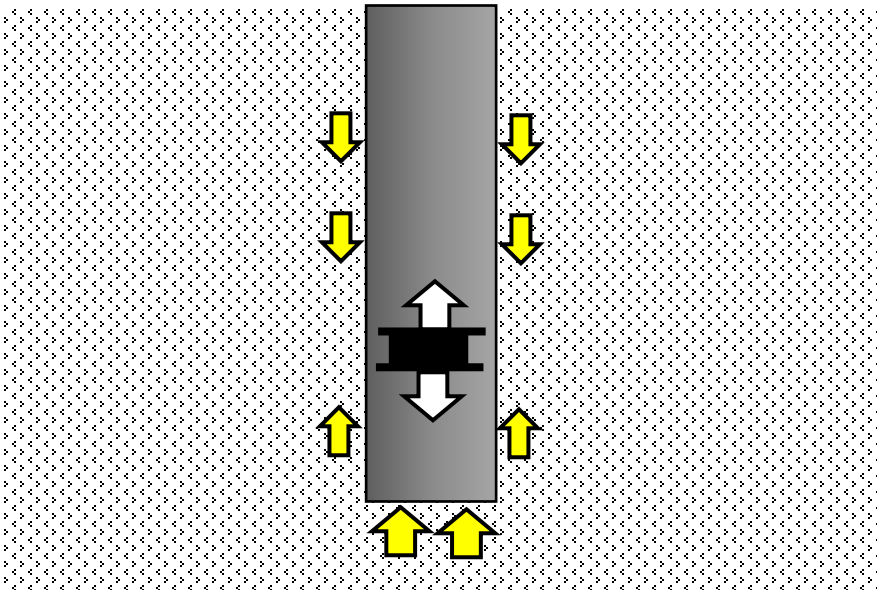


Figure 13-14 Bi-Directional Testing Schematic

The maximum test load is limited to the maximum axial resistance of the shaft above or below the cell, or the maximum capacity of the load cell, or the maximum expansion of the load cells (typically 6 inches). It is therefore important that the load cell be located at or near the point in the shaft where the axial resistance above and below the cell are approximately equal. If the side resistance exceeds the base resistance, then the load cell should be located at the point at which the base resistance plus the side resistance below the load cell are approximately equal to the side resistance above the load cell. If the base resistance exceeds the side resistance, then the magnitude of the base resistance that can be mobilized during the test is limited as illustrated in Figure 13-15. The latter limitation can be mitigated in some cases using the "Chicago Method" described previously and illustrated on Figure 13-10. The designer planning the load test must make estimates of the magnitude of the resistance above and below the cell, and the successful measurement of maximum resistance values is thus dependent on the ability of the designer to place the cell in the optimum location.

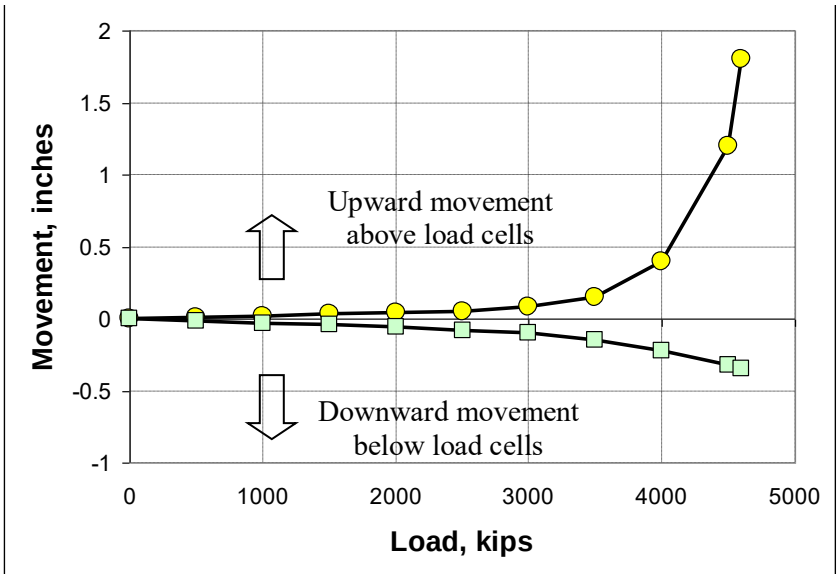


Figure 13-15 Example Test Result from Bi-Directional Test

On one recent project at a major bridge site in Louisiana, the base resistance (around 3,000 tons) exceeded the available side shear resistance (around 2,000 tons) and limited the ability of the load cell loading method to mobilize and verify the magnitude of the available base resistance. The contractor employed a 1,000-ton capacity reaction frame with a hydraulic jack atop the test shaft as shown in Figure 13-16. When the upward shaft movement indicated that the side shear was at its maximum limit, the load cell loading was paused, and the reaction system engaged. The jack at the top of shaft was loaded simultaneously with the increased load on the load cells so that the base resistance was successfully mobilized using both the side shear of the shaft plus the reaction system.

Another possible approach to load cell testing is to use cells at multiple levels, as illustrated on Figure 13-17. By performing this "multiple stage" loading, the resistance of various segments can be measured separately and the limitation of the "weakest link" can be overcome.

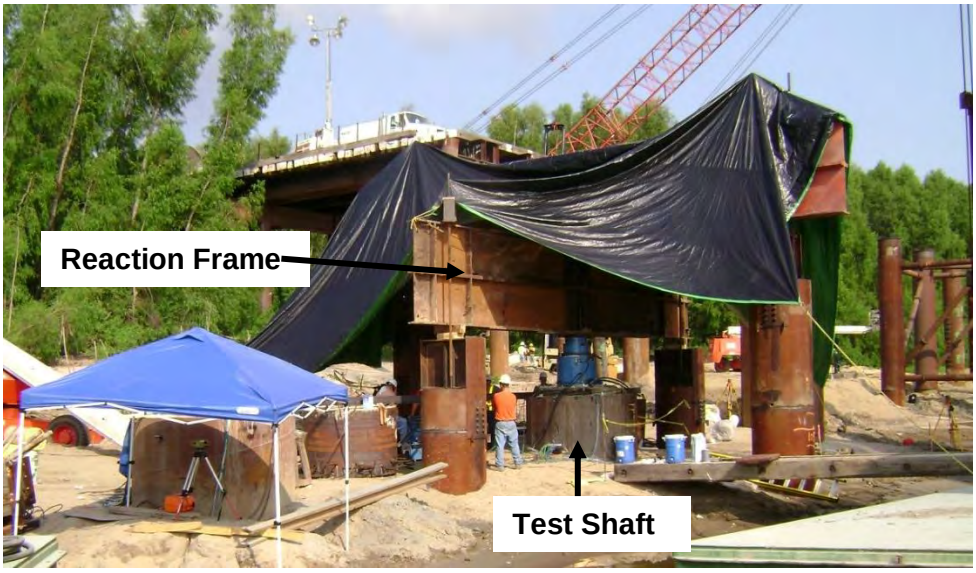


Figure 13-16 Bi-Directional Load Test with Added Reaction System

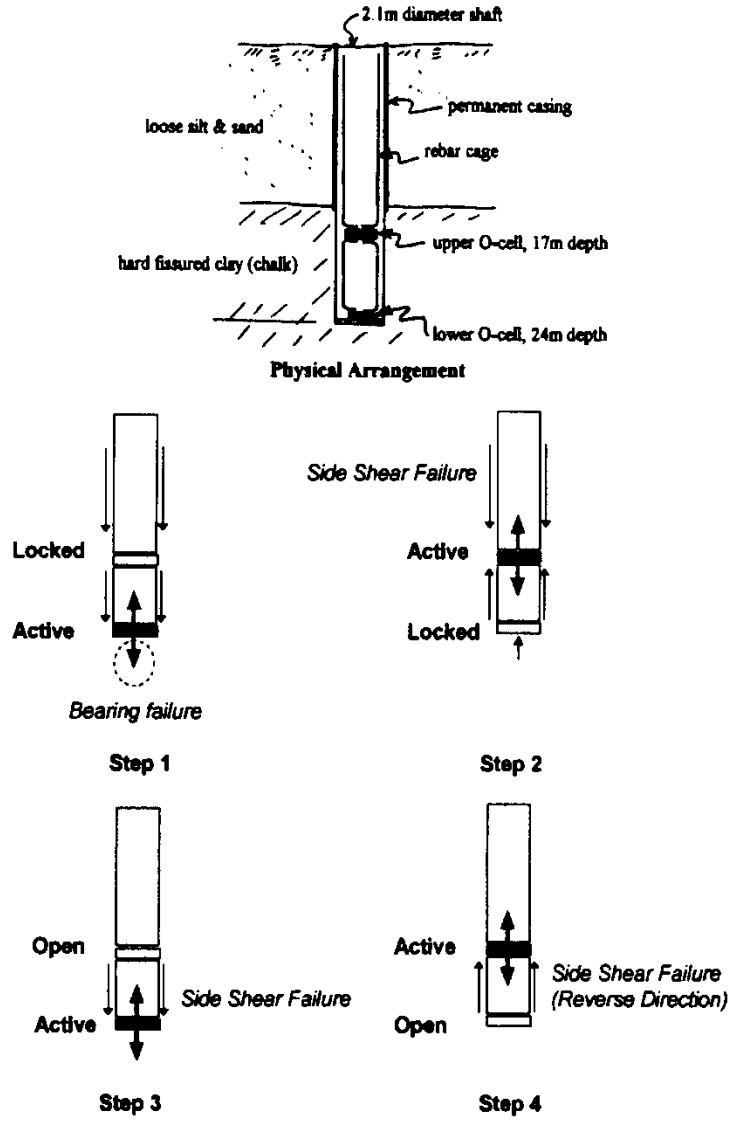


Figure 13-17 Multiple-Level Arrangement for Bi-directional Load Cells (O'Neill et al., 1997)

The ability to employ the bi-directional testing technique in production shafts is limited, unless the load cell is located near the bottom of the drilled shaft. Since the cells must crack and separate the various sections of the shaft, bending moments cannot be transferred below the uppermost load cell level. In addition, the large upward displacement of a section of shaft above the load cell could be detrimental to the subsequent axial resistance of that portion of the shaft to a downward directed load in some cohesive soils or rock materials. Measurements of axial resistance during load reversal of segments between multiple level load cells suggest that a non-recoverable loss of axial resistance in side shear may be observed with full cyclic stress reversal at displacements greater than about 1/2 inch. This phenomenon is likely related to remolding of cemented materials at the rough shaft/soil interface and has not been reported with uncemented granular soils. For these reasons, it is preferred that bi-directional load cell tests be performed on dedicated load test shafts so that tests can be conducted to achieve the objective of measuring axial resistance without concern about the subsequent use of the test shaft. If bi-directional tests are performed on production shafts, it is recommended that the upward movement be limited to about 1/2 inch during testing in cemented or cohesive soils or rock to minimize potential degradation related to cyclic stress reversal.

The following sections describe the loading apparatus and procedures used for performing a bi-directional load test and for determining the static axial resistance of a drilled shaft from the measurements.

13.2.2.2.1 Loading Apparatus and Procedure

Bi-directional load cells are available in a range of sizes and load capacities but are often assembled with multiple load cells at a single level to provide extremely high forces. For example, a recent bridge project in St. Louis (Axtell and Brown, 2011) included a bi-directional load test on an 11-ft diameter shaft in limestone in which the combination of cells provided a force of over 36,000 kips, thereby mobilizing a total axial resistance of over 72,000 kips. The limit to the amount of load that can be delivered is the cross-sectional space available for load cell placement and the maximum hydraulic pressure that can be applied to each cell. When multiple load cells are utilized per level, it is advisable to utilize a minimum of three per level to alleviate concerns of load eccentricities and of shaft flexure causing damage to the cells. Multiple load cells also provide the advantage that the tremie pipe for concrete placement can go through the center of the assembly rather than to the side. Typical load cell assemblies are illustrated in Figure 13-18: the single cell on the left was positioned at the base of a dry excavation and set into a bed of fluid concrete prior to filling the remainder of the excavation; the multiple cell assembly on the right includes four smaller cells and a funnel shaped rebar pattern to provide a guide for the tremie pipe to pass through the center of the load cell assembly.



Figure 13-18 Single and Multiple Load Cell Assembly

Load cells typically have an available travel (separation) of approximately 6 inches before the cells lose their seal; actual travel available is to some degree affected by the amount of eccentricity imposed to the cell.

The upper and lower plates for the load cell assemblies are typically around 2 inches thick, and have generous portions cut away between the load cell locations to allow for concrete flow through the plates during concrete placement operation of shaft construction. The plates utilized are subject to the specific load conditions and geo-material present at the site being tested. The outside diameter of the load plates are typically sized such that they just fit inside the vertical rebar of the cage.

There must be no reinforcing across the plane where the shaft cracks into two segments (load cell break plane) prior to final placement of the cage into the excavation. Often some of the rebar, or other structural steel welded in place, is left intact across the load cell plane rather than relying solely on the tack welds (holding the load cells closed) to facilitate flexural stresses experienced during lifting of the cage. The load cell assemblies make the cage much stiffer at these locations, and can be subject to damage if distortion occurs when the cage is lifted. This temporary reinforcement is removed once the cage is vertical.

Cross-hole sonic logging (CSL) tubes that pass the load cell levels are typically scored to allow them to rupture when the load is applied to the load cells.

The bi-directional load applied to the drilled shaft is usually monitored by measuring the pressure applied by the pump. The load cells will therefore need to be calibrated in a testing machine prior to installation to obtain a relation between the measured pressure and the load applied by the cell. In practice, the hydraulic pressure will usually be measured at the ground surface, but the cells are typically located at substantial depths below the ground surface (30 ft to over 200 ft). Therefore, the actual pressure at the level of the cell is the pressure that is measured plus the vertical distance from the pressure gauge to the middle of the cell times the unit weight of the fluid. This correction, while conservative if ignored, may be made before plotting load versus movement.

Displacements of the various segments being loaded in a bi-directional test must be measured separately. Separation of the load cells (relative movement of the upper shaft section to the lower) is measured by expendable electronic displacement transducers that are placed between the load cell plates at multiple locations around the shaft and cast into the shaft during concreting operations. "Telltales" are sleeved, unstrained rods extending to the top of the shaft from the upper and/or lower plates, and are used to determine the movement of the load cells relative to the top of the shaft. To determine the absolute movement of the shaft, the top of shaft displacement is measured relative to a stable reference. The telltales measure shaft compression directly, and the top of load cell movement is the top of shaft movement summed with the shaft compression. With long cages and/or cages set in multiple sections, often only the top section is fitted with a conventional telltale that "daylights" out of the top of the shaft. Deeper shaft sections may include an embedded telltale with a sacrificial displacement transducer. Strain gages are also embedded at selected elevations within the shaft to provide measurement of load transfer at various points along the shaft. A discussion of measurements and instrumentation is provided in Section 13.2.3.

13.2.2.2.2 Derivation of Static Axial Resistance

The bi-directional load test measurements provide a measure of the load versus displacement response of separate segments of the test shaft. Each segment is loaded in static compression, and at least one segment is loaded in the upward direction. Typical loading procedures apply load in intervals of 5% to 10% of the anticipated nominal axial resistance of the test segments, with load increments maintained for a period of 4 to 8 minutes and measurements made at 1, 2, 4 and 8 minute intervals.

Interpretation of bi-directional load test results on an individual segment pushed down are performed in a manner similar to any other static load test measurement. Interpretation of measurements on a segment pushed upward are similar except that the dead weight of the shaft segment is subtracted from the load cell load to obtain a net upward load applied to the soil. Strain gauge measurements may be used to interpret load distribution within each segment, as outlined in Section 13.2.3.

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

$$f_s = \frac{Q_{oc}}{\pi BD} \qquad 13\text{-}2$$

where: f_s = average unit side resistance (stress)
Q_{OC} = bi-directional test load
B = socket diameter
D = socket length above the load cell base

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

Bi-directional load cell test results may be used to construct an equivalent top-loaded settlement curve, as illustrated in Figure 13-19. At equivalent values of displacement, both components of load are added. For example, in Figure 13-19a, the displacement for both points labeled '4' is 0.4 inches. The measured upward and downward loads determined for this displacement are added to obtain the equivalent top load for a downward displacement of 0.4 inches and plotted on a load-displacement curve as shown in Figure 13-19b. 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 13-19a, 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 13-19a, this is the base resistance curve). The resistance provided by the other section is estimated in this example by extrapolating its curve beyond the maximum test load. In Figure 13-19, the side resistance curve is extrapolated. The resulting equivalent top-loaded settlement curve shown in Figure 13-19b is therefore based on direct measurements up to a certain point, and partially on extrapolated estimates beyond that point. Elastic compression of the shaft above the test segments may be computed and added to the measured displacements.

For design using the higher resistance factors provided in AASHTO design guidelines based upon confirmation by load tests, it is not appropriate to use extrapolated values of side resistance beyond the maximum mobilized values confirmed by the load test(s). If ductile behavior in side shear is anticipated with no strain-softening (reduction in side resistance at larger displacements), then the strength limit is confirmed by the maximum values achieved during the load test. For the above example, this approach would effectively assume that the shaft side shear is limited to the value shown at point 5 in Figure 13-19a and that the extrapolated curve above point 5 is a vertical line. Where the test shaft above the cells is the limiting factor (as illustrated in Figure 13-15) and the lower portion is not fully mobilized, the test has verified the magnitude of resistance that is mobilized during the loading.

Because of the tendency for dilatancy at the rock/shaft interface, it is unusual for strain softening behavior to be observed in load tests of drilled shafts. However, if the load test data are similar to Figure 13-19 where side shear at large displacements are not measured, the test does not provide a direct measure of the combined performance of side shear and base resistance at large displacement. In the event that strain softening behavior in side shear is a potential concern, it may be unconservative to extrapolate the side shear resistance measurements at limited displacement. In this case, the strength limit in base resistance may be at a displacement which is incompatible with the displacements at maximum side shear, and therefore the expected top-loaded settlement curve would need to be adjusted for a reduction in side shear beyond point 5 in Figure 13-19a. If strain softening behavior is suspected, it is important that test data be obtained at large side shear displacements so that the effect can be observed and evaluated.

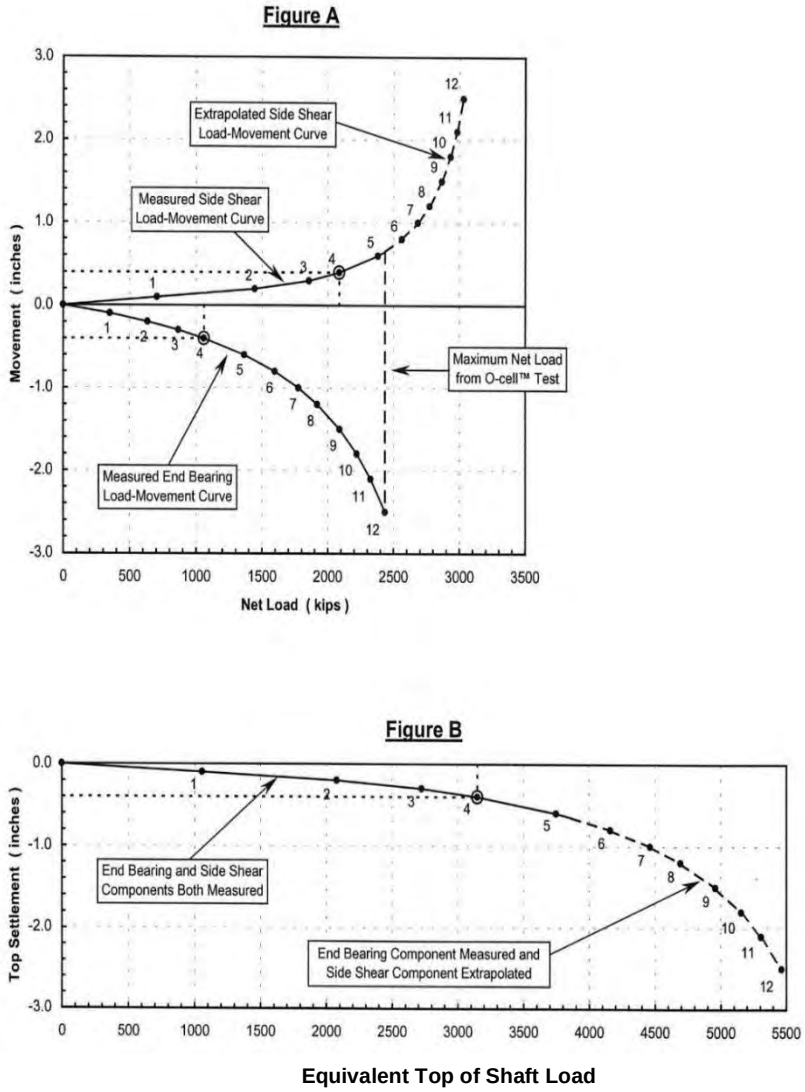


Figure 13-19 Construction of Equivalent Top-Loaded Settlement Curve from Bi-directional Load Test Results (Figures courtesy Loadtest, Inc.)

According to Paikowsky et al. (2004), most state DOT geotechnical engineers utilizing bi-directional load testing tend to accept the measurements as indicative of drilled shaft performance under conventional top-down loading. Bi-directional load test results are applied in design by construction of an equivalent top-load settlement curve, as illustrated above, perhaps by using the measured unit side and base resistances as design nominal values where the test is used as site specific verification for justification of the higher resistance factors for design. There is little evidence that drilled shafts deriving axial resistance in soil exhibit any significance difference in behavior associated with direction of loading. Figure 13-20 illustrates comparative data from the bi-directional and Kentledge (top-down) load tests that were illustrated in Figure 13-1 on two identical shafts in sandy clay soil at a site in Singapore (Molnit and Lee, 1998).

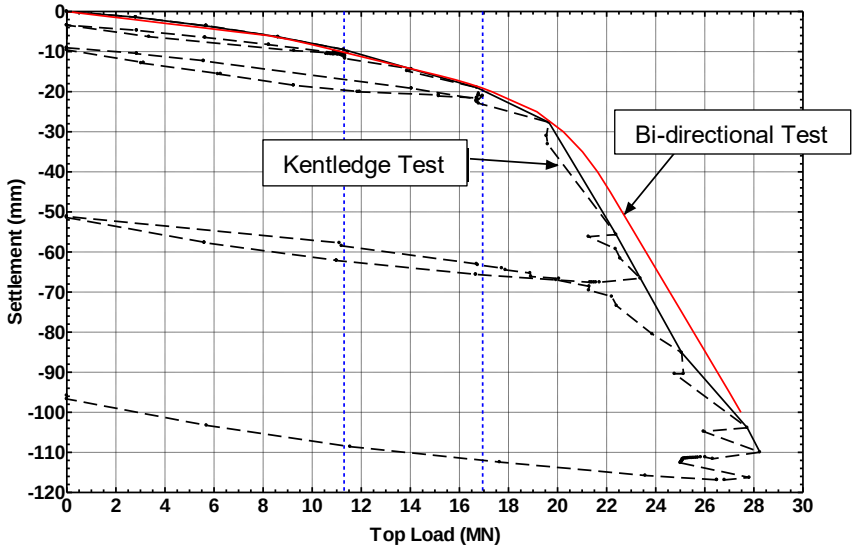


Figure 13-20 Top Down and Bi-directional Load Test at a Soil Site in Singapore (1 inch = 25.4 mm; 1MN = 112 tons)

In shallow rock sockets under bottom-up loading conditions, a potential failure mode is by formation of a conical wedge-type failure surface ("cone breakout"). Obviously, 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 13-21a, while loading from a bi-directional load cell at the bottom of the socket (Figure 13-21b) would lift the shaft from the bearing surface. The effect could occur where rock is fractured near the top of the formation due to weathering or other effects.

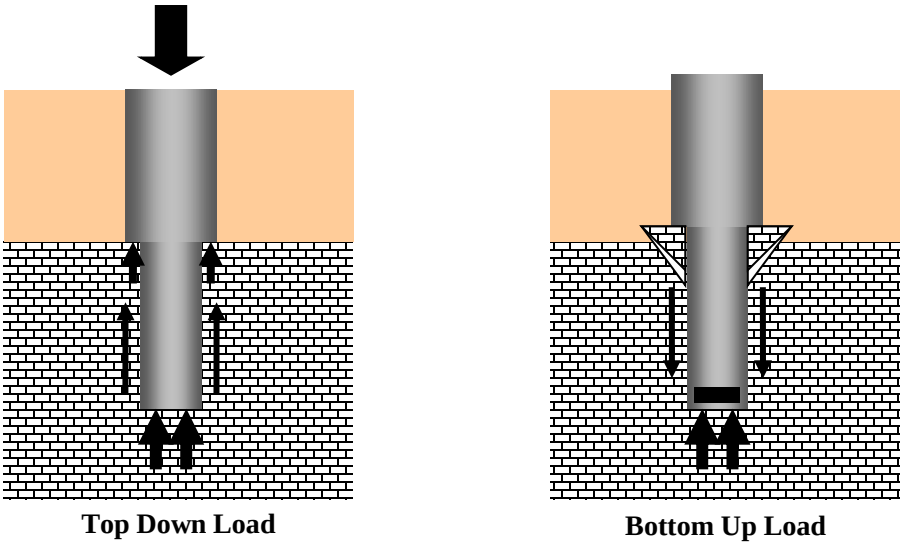


Figure 13-21 Effect of "Shoulder" at Top of Rock: a) Top-Down Loading; b) Bottom-Up Loading

Although full scale comparative test data are not available for drilled shafts in rock, some researchers (McVay et al., 1994; O'Neill et al., 1997; Shi, 2003) have pointed out differences between bi-directional load test conditions and top loading conditions in rock that may require interpretation. The most significant difference is that compression 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 as shown in Figure 13-22. Dilatancy at the shaft/rock interface adds to the effect, with the result that the normal stress at the shaft/rock interface may be less in the bi-directional load test than in a top-down load test. Loading from an embedded load 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 load 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 upon subsurface conditions, different total side resistance of a rock socket. Varying load transfer distribution with direction of loading computed using a finite element model are illustrated in Figure 13-23.

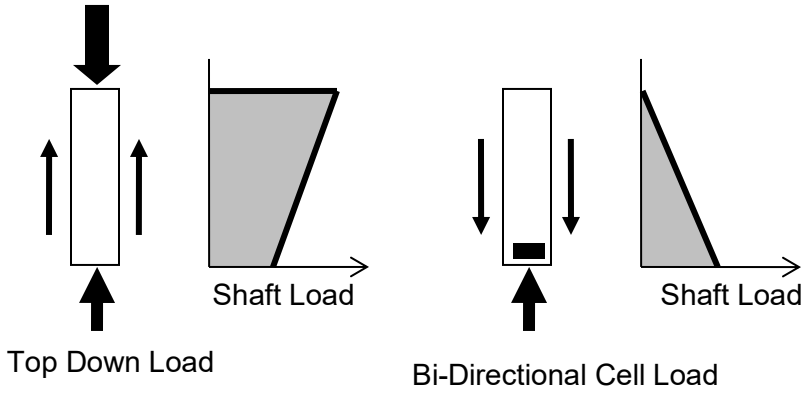


Figure 13-22 Average Compressive Load in Shaft During Top Down and Bi-directional Loading

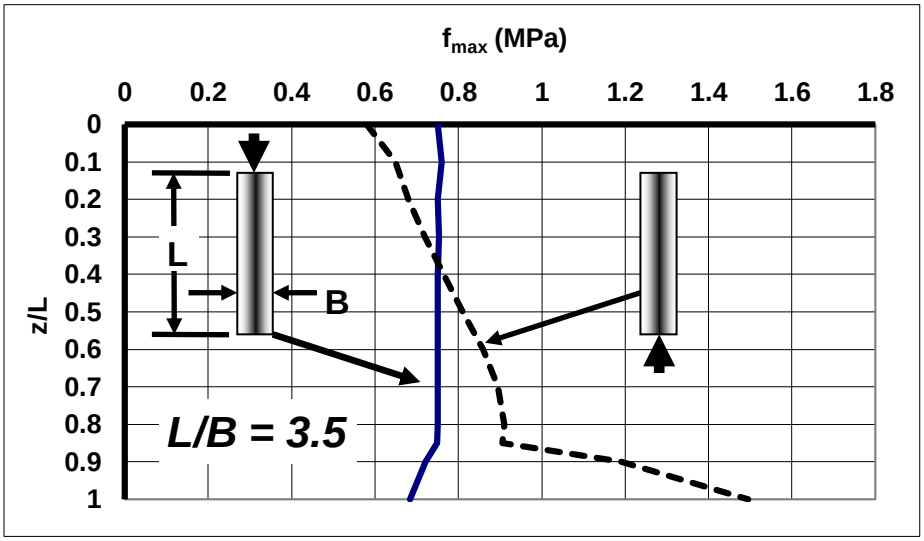


Figure 13-23 Analytical Model Results for O-Cell Loading in a Rock Socket (McVay et al, 1994)

Finite element modeling (FEM) reported by Shi (2003) suggests that differences in rock end bearing response between bi-directional load testing and top-load testing were not significant, and that differences in rock side shear response may be affected by (1) modulus of the rock mass, E_M, and (2) interface friction angle. Shi first calibrated the FEM model to provide good agreement with the results of bi-directional load tests on full-scale rock socketed shafts, including a test shaft socketed into shale in Wilsonville, AL, and a test shaft in claystone in Denver, CO, described by Abu-Hejleh et al. (2003). In the FEM model, load was applied similarly to the field bi-directional load test, i.e., 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 to the equivalent top-load settlement curve determined from bi-directional load test results. Figure 13-24 shows a comparison of the top-load versus displacement curves for the Alabama test, one as calculated from the bi-directional load test and the other as predicted by FEM analysis using material properties back-fitted to the bi-directional load test. The curves show good agreement at small displacement, but the curve derived from FEM analysis is stiffer at higher displacement.

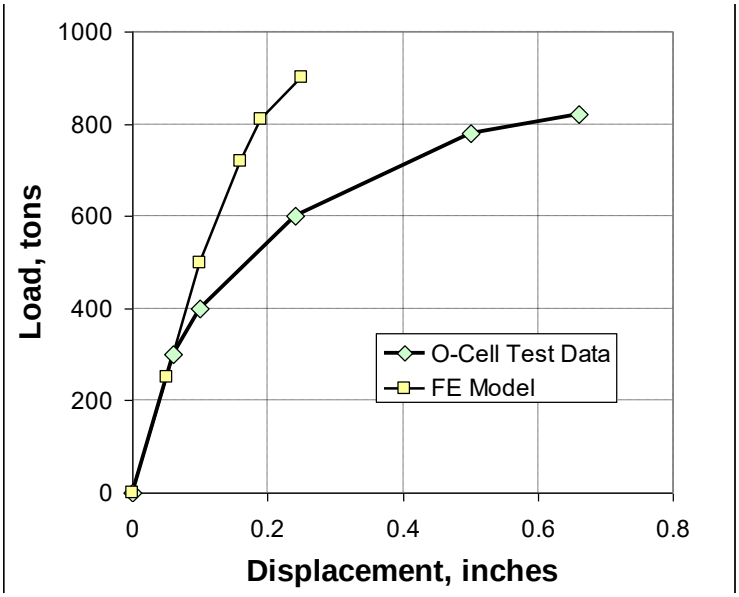


Figure 13-24 Analytical Model Results for Bi-directional Loading in a Rock Socket (Shi, 2003)

These analytical models suggest that the equivalent top-load settlement curve derived from a bi-directional load test may underestimate side resistance relative to the situation in which end bearing resistance is simultaneously engaged and including higher axial displacements, *i.e.*, the derived curve is conservative. The differences between loading from the bottom and loading in compression from the top are due to differing normal stress conditions at the shaft/rock interface, and 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 bi-directional loading and top-down compression loading may warrant consideration in some cases.

Based on the available information, the following general guidelines are suggested for interpretation of bi-directional load tests:

- The measured axial resistance in end bearing and side shear for shafts in soil is equal to top-down loading.
- The measured load transfer in end bearing for shafts in rock is similar to top down loading.
- The measured load transfer in side shear for shafts in rock, on average, may be conservative with respect to top-down loading.

- The distribution of load transfer in side shear for shafts in rock may be biased toward higher unit values in the lower portion of the socket relative to the upper portion. Caution should be exercised with respect to extrapolation of the measured side shear resistance in the lowermost portion of the rock socket (presumably based on strain gauge data) to sockets of greater length. Unusually high unit side shear values in the lowermost portion of the socket may be an artifact of the bottom-up loading, and average values may be more representative of shaft performance.
- Construction of the equivalent top load-settlement curve should include an adjustment for the elastic compression of the drilled shaft.
- For design purposes, a careful interpretation of the results of a bi-directional load test should provide a measurement which is considered equivalent to a static load test. Therefore, the resistance factors appropriate for design based on static load test results should apply.

Interpretation of bi-directional load test data and unit values of axial resistance most often is based on strain gauge instrumentation. Additional information on strain measurements and instrumentation is provided in Section 13.2.3.

13.2.2.2.3 Advantages and Limitations of Bi-Directional Load Testing

Some of the advantages of the bi-directional loading for axial load testing of drilled shafts include:

- The test provides the ability to apply larger loads than any of the available methods. This feature is especially important for rock sockets.
- The large capacity allows testing of production-size shafts, in most cases.
- With multiple cells or proper instrumentation, the base and side resistances are isolated from the resistance of other geomaterial layers.
- The loading is static and can be maintained to observe creep behavior.

Limitations of the bi-directional load test include:

- The shaft to be tested must be pre-determined so that cells can be included; it is not possible to test an existing shaft in which bi-directional load cells have not been pre-installed.
- For each installed device, the test is limited to failure of one part of the shaft, unless multi-level cells are used.
- The performance of a production shaft subject to top-down loading must be computed, and may require extrapolation of data in some cases.
- Limitations exist related to using a test shaft as a production shaft.
- The effect of upward directed loading compared to top-down loading in a rock socket is not completely understood. Current procedures for estimating side resistance in rock sockets are likely to be conservative.

The bi-directional load test provides transportation agencies with a practical and cost-effective tool for evaluating the performance of high capacity drilled shafts and it is expected that the bi-directional load test will continue to be used extensively. In many cases, it is the only practical method for field measurement of performance due to the magnitude of the loads imposed.

13.2.2.3 Rapid Load Test

Rapid loading utilized for testing of drilled shafts and other deep foundation elements employs a load which is so rapid that inertial and damping effects are important, but of sufficient duration that wave propagation effects are minimal. The test shaft is thus subjected to a fast "push" rather than a sharp blow as might be observed from a pile driving hammer. A relative comparison of the load applied to the shaft versus time is illustrated on Figure 13-25. With a rapid loading, the load is applied for a sufficient time interval that the shaft is pushed downward with the top and base of shaft moving together in a nearly rigid body motion. A hammer impact typically induces a short duration compressive stress wave which propagates down the shaft from the top to the base and returns as a reflected wave. The latter is referred to as a high strain dynamic loading and is described in Section 13.2.2.3.4. The blow of a high strain dynamic test typically involves deceleration of a mass (the ram) which has a weight equal to about 1% of the test load. The load pulse of a rapid load test typically involves a mass which has a weight equal to about 5% to 10% of the test load.

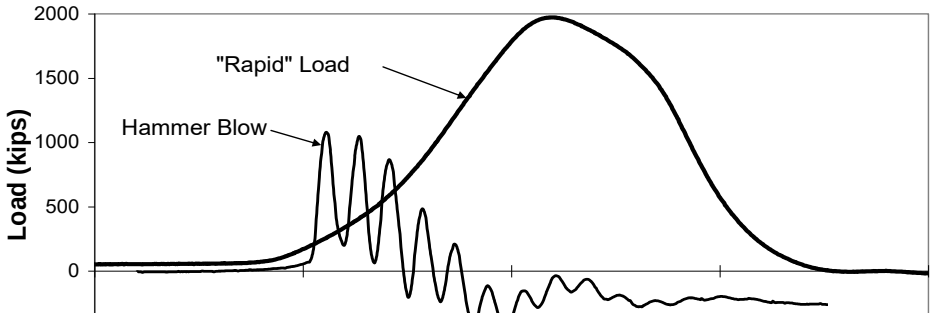


Figure 13-25 Comparison of Rapid Loading and Dynamic Loading

There are two methods of initiating a rapid loading on the top of a drilled shaft for purposes of testing, both of which are described in ASTM D-7383. One method is to drop a heavy mass onto the top of the shaft, but to use such a soft cushion between the mass and the shaft that the deceleration of the mass occurs over the desired time interval. Such a cushion could consist of springs with an appropriate stiffness. Although the dropped mass method has been reportedly used to generate forces of up to 500 tons on drilled shafts, the more common method is to position a heavy mass onto the top of the shaft and accelerate the mass upward using combustion gas pressure.

The test using combustion gas pressure is Newton's 2nd Law in action, Force = mass x acceleration. When the reaction mass is accelerated upward with 20g's of thrust, an equal and opposite downward force of 20 x the weight of the mass is applied, as illustrated on Figure 13-26.

The Statnamic® loading device uses combustion gas pressure to produce this type of loading and was developed in the late 1980's and early 1990's by Berminghammer Foundation Equipment of Hamilton, Ontario (Bermingham and Janes, 1989). 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 a device owned by FHWA for research purposes.

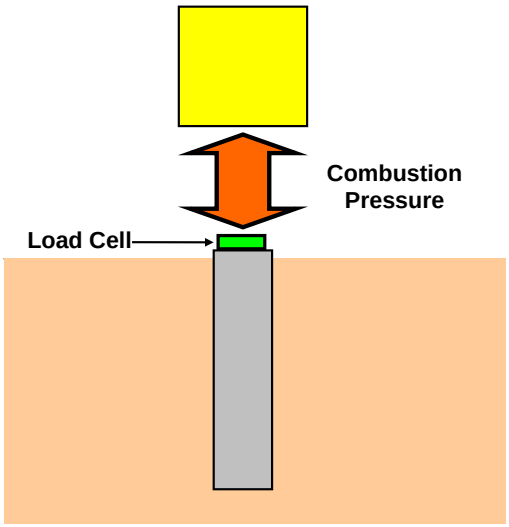


Figure 13-26 Conceptual Sketch of the Rapid Load Test using Combustion Gas Pressure

Currently available loading systems provide a means to apply loads of up to 5,000 tons to the top of the shaft without the requirement for an independent reaction system, achieved by a device such as the one illustrated in Figure 13-27. Combustion occurs in a central chamber and "launches" the reaction masses upwards to a height of about 8 to 10 ft. The 2,000-ton capacity system shown in Figure 13-27 employs a "catch" frame mechanism to contain the reaction masses in the center of the device. A 5,000-ton capacity device (which employs a gravel containment system to support the reaction masses) represents the upper limit of the load capacity of this system at present (2018).

The remainder of this section will present an overview of rapid load testing as implemented with the combustion gas pressure device, which is currently the more common application of rapid load testing for drilled shaft foundations in North America.



Figure 13-27 Rapid Load Testing Device (2000 Ton Capacity)

13.2.2.3.1 Loading Apparatus and Procedure

The upward acceleration of the reaction mass is achieved using combustion of pelletized fuel (similar to the material used in automobile air bags) to generate gas pressure within a piston. The downward force is directed through a calibrated load cell which provides measurements of the applied force as a function of time. Servo-accelerometers mounted on the top of the shaft provide measurement of the acceleration and displacement (by integration of the acceleration).

Example measurements of load, displacement, and acceleration are illustrated on Figure 13-28. The measured acceleration times the weight of the shaft represents an inertial force which acts counter to the applied force measured via the load cell; the actual force resisted by the soil is the difference between these two forces. The downward movement of the shaft occurs as this force pushes the shaft into the soil. The test shaft illustrated in Figure 13-28 shows a small rebound at the end of the loading period, and most of the 2+ inches of movement is permanent displacement.

The behavior of the test shaft is illustrated by the load cell and load interpreted from strain measurements on a drilled shaft shown in Figure 13-29. The shaft was socketed into limestone between elevation +26 ft and the base of the shaft at elevation +16 ft. The load cell was at the top of the 40-ft long shaft at elevation +56 ft. The strain data indicate that the behavior of the shaft during loading was similar to that of a rigid body. The travel time for a compression wave in a 40-ft long concrete member is approximately 0.003 seconds. This delay in the arrival of the peak force at elevation +19 ft relative to the load cell is small compared to the duration of the load pulse.

The following section describes the interpretation of the measurements to obtain static axial resistance. Instrumentation is described in Section 13.2.3.

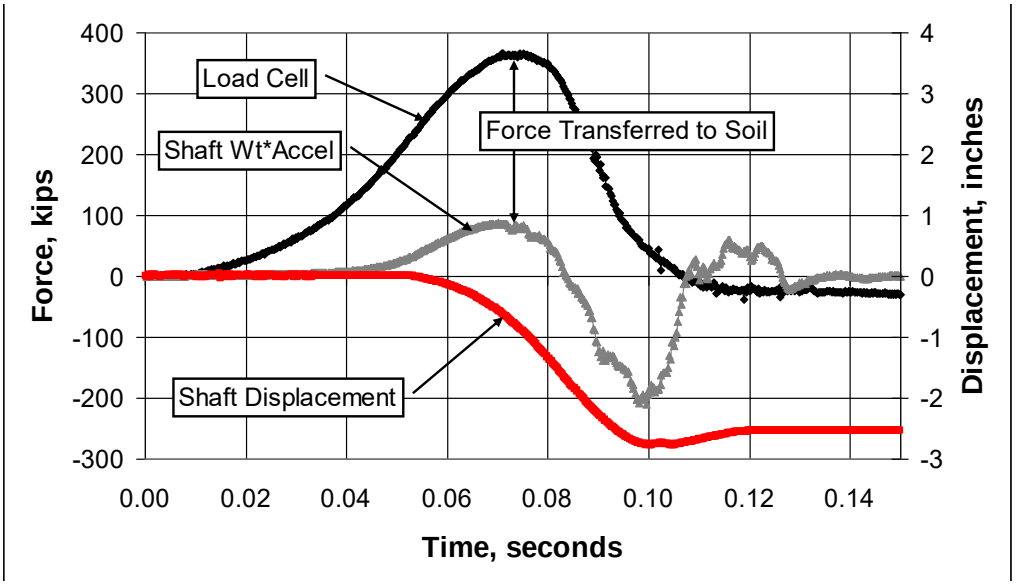


Figure 13-28 Measurements of Force and Displacement During Rapid Loading

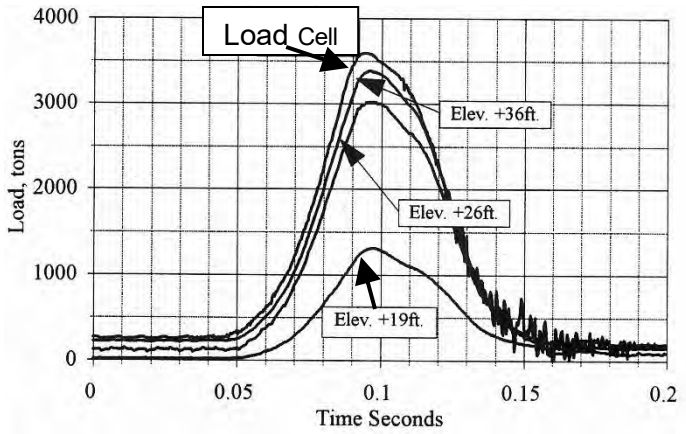


Figure 13-29 Force-Time Measurements for a Drilled Shaft

13.2.2.3.2 Derivation of Static Axial Resistance

The "unloading point method" (UPM) represents a simple method of analysis of the results of a rapid load test which can be used in many cases. The UPM also provides the basis for more complex analyses described subsequently. Because of the duration of loading, all elements of the pile move in the same direction and with almost the same velocity; the UPM, therefore, is a simple approach which treats the shaft as a rigid body undergoing translation. Analytical studies by Brown (1994) have shown that this rigid body assumption is not appropriate for "long" shafts (generally more than about 80 to 100 ft in length). Subsequently, a more rigorous method of analysis called the "segmental unloading point method" (SUPM) has been developed by Mullins et al. (2002).

The forces acting on the shaft during a rapid load test include the applied load, F_{stn}, the shaft inertia force, F_a, and the soil resistance forces. The applied force, F_{stn}, is measured. The shaft inertia force, F_a, is equal to the mass of the shaft times the acceleration of the shaft, where the mass is known and the acceleration is measured. Because of the rapid loading rate, the soil resistance forces include both the static soil resistance, F_u, and the dynamic soil resistance, F_v. The dynamic resistance includes any forces which might be rate-dependent and not mobilized during a conventional static load test, such as resistance from transient pore water pressures and other dynamic components of resistance. This component of force is modeled as a viscous damper, with a force that is proportional to the velocity. A simple single degree of freedom model of the problem is illustrated in Figure 13-30.

In mathematical terms, the force equilibrium on the pile may be described as follows:

$$F_{stn}(t) = F_a(t) + F_u(t) + F_v(t) \qquad 13\text{-}3$$

This equation may be rewritten in terms of static soil resistance as follows:

$$F_u(t) = F_{stn}(t) - F_a(t) - F_v(t) \qquad 13\text{-}4$$

Because the dynamic resistance in the model is proportional to velocity, this force is zero at the point where the shaft velocity is zero. The point of zero velocity is referred to as the "unloading point", and is indicated as point 2 on Figure 13-31. At the unloading point, Equation 13-4 can be solved for F_u because F_{stn} and F_a are known.

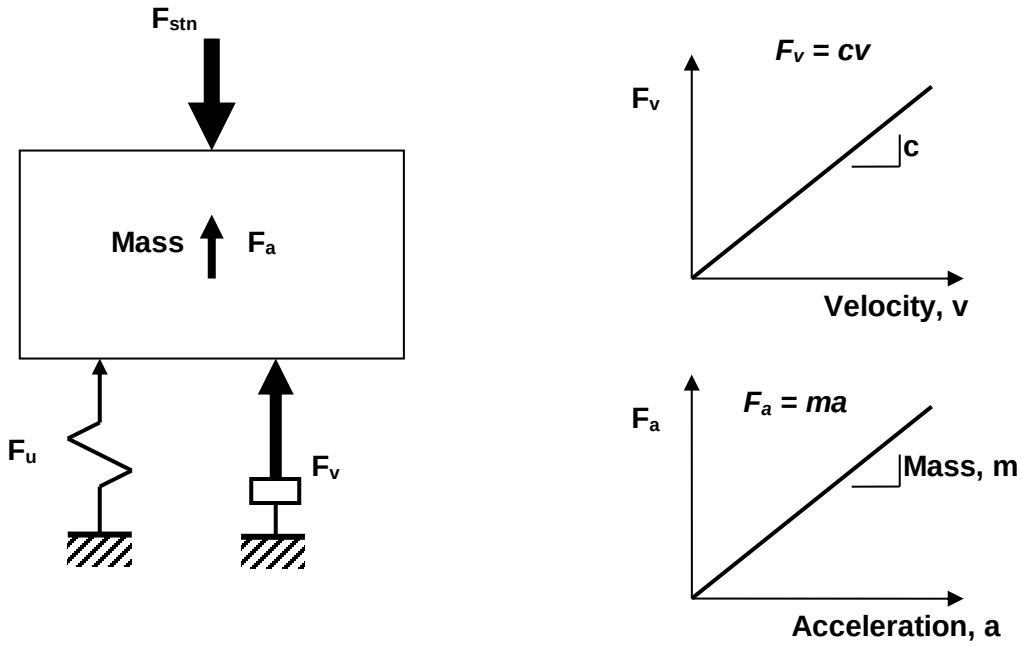


Figure 13-30 Single Degree of Freedom Model of a Rapid Load Test

The UPM derives the static resistance over the remainder of the curve as follows. The maximum static resistance is assumed to be mobilized and constant over the portion of the curve between the peak applied force (point 1 in Figure 13-31) and the unloading point. This portion of the curve is then used to compute the damping coefficient, c, as indicated in Equation 13-5.

$$c(t) = \frac{F_{stn}(t) - F_a(t) - F_{u\max}}{v(t)} \qquad 13\text{-}5$$

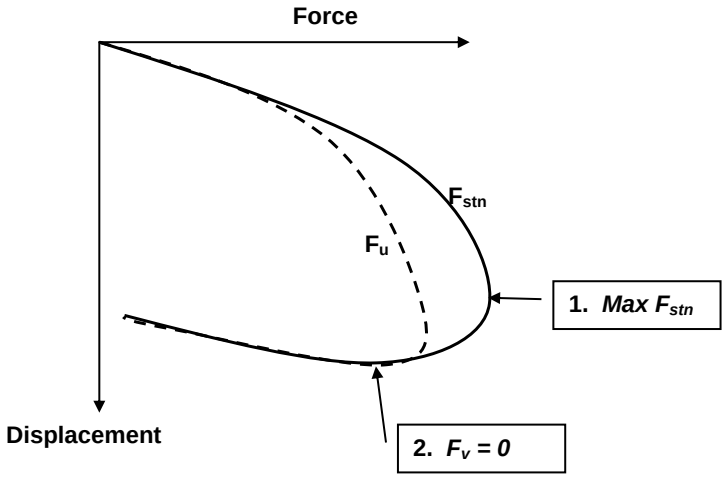


Figure 13-31 Rapid Test Load versus Displacement

The damping coefficient, c, is directly computed at each measurement point between points 1 and 2, a best estimate average value is chosen as illustrated in Figure 13-32. This constant is then used to derive the remainder of the static load versus displacement curve using Equation 13-4, and illustrated on Figure 13-31. A slight modification of the above procedure may be made by using the computed or measured average acceleration of the shaft instead of assuming a rigid body motion.

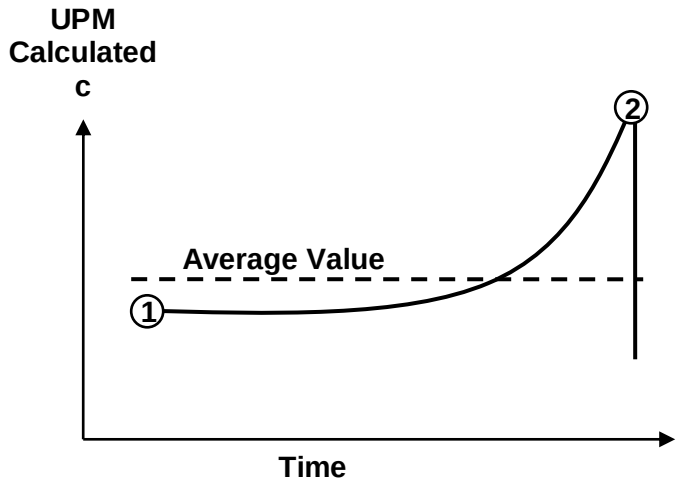


Figure 13-32 Computed Damping Coefficient, c

For long shafts, the "segmental unloading point method" (SUP) is used. This method is based on the UPM described above but treats the shaft as a series of segments as illustrated in Figure 13-33. Instrumentation within the shaft is typically used to determine the forces transferred between segments during the combustion gas pressure test.

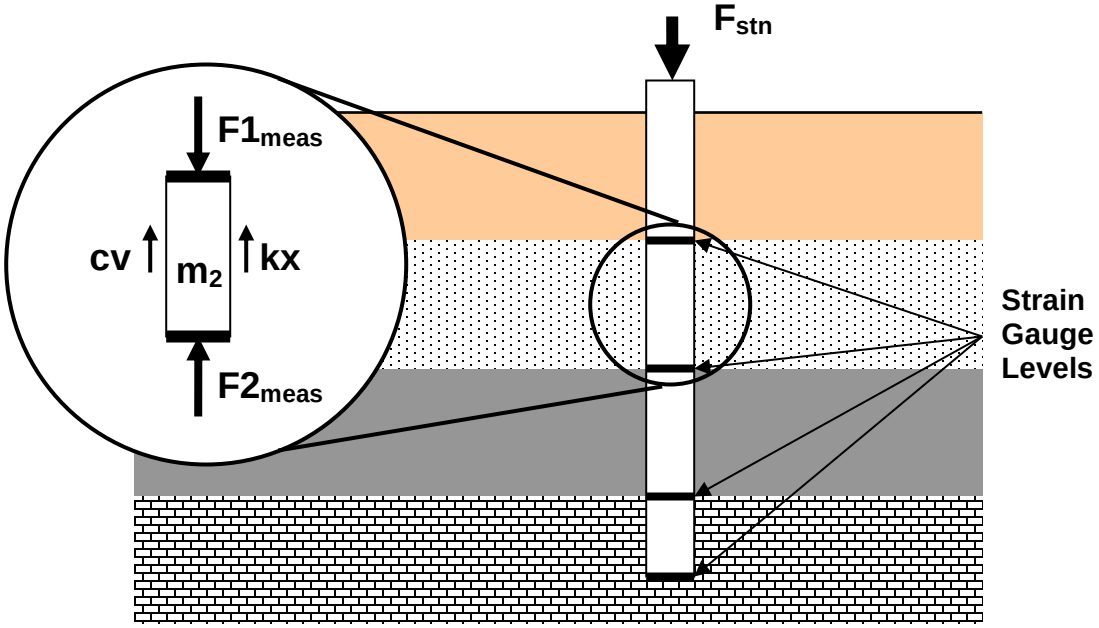


Figure 13-33 Segmental Unloading Point Method

Although the static resistance derived from the rapid load test as described above has accounted for dynamic effects during the test, there remains a rate-of-loading effect in the static resistance because the duration of the test is short compared to conventional static loading. Therefore, an empirical rate effect parameter is applied to the derived static resistance based on comparative tests described by Mullins (2002) by multiplying the static resistance by the factors shown in Table 13-4.

TABLE 13-4 RATE FACTORS FOR DIFFERENT SOIL TYPES

ROCK	SAND	SILT	CLAY
0.96	0.91	0.69	0.65

Comparative data between combustion gas pressure load tests and static load tests on drilled shafts in Florida are reported by McVay et al. (2003) and illustrated in Figure 13-34. For design of drilled shafts calibrated to site-specific rapid load test data, Florida DOT guidelines recommend a resistance factor of 0.70 for shafts with redundancy and 0.60 for non-redundant drilled shafts (compared to 0.75 and 0.65 for static load tests). Note that AASHTO Section 10.5.5.2.4 provides for a maximum resistance factor of 0.70 for drilled shafts calibrated to site-specific static load tests, with a 20% reduction in resistance factor for a non-redundant shaft foundation such as a single shaft supporting a single column.

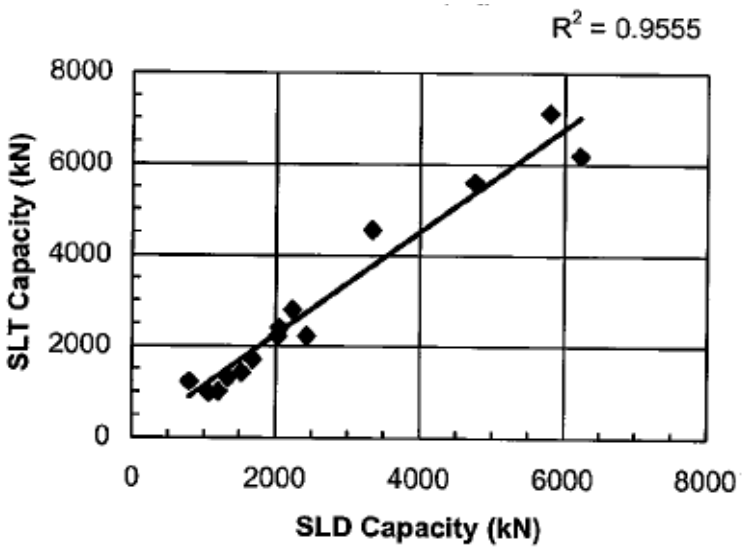


Figure 13-34 Comparative Static (SLT) and Rapid (SLD) Load Tests for Drilled Shafts (from McVay et al., 2003)

Considering that some additional variability is introduced by performing a rapid load test compared to a conventional static load test, it is recommended that general design of drilled shaft foundations where site-specific rapid load test data are calibrated to static load test data should utilize a resistance factor which is 0.05 less than the factors outlined in AASHTO Section 10.5.5.2.4 for design based on static load tests. Therefore, a maximum resistance factor of 0.65 is indicated for design of drilled shafts calibrated to site-specific rapid load tests, with lesser values for non-calibrated rapid load tests, non-redundant foundations or unusually high site variability.

13.2.2.3.3 Advantages and Limitations of Rapid Load Testing

Most of the advantages and limitations of rapid load testing using the rapid load testing device are apparent from the previous discussion. Loads of up to 5,000 tons can be applied to the top of a drilled shaft foundation without the need for a reaction system or without special preparations during construction. These capabilities generally make this a cost effective tool which is useful for proof tests on production foundations or for performing multiple load tests at a site. The device is particularly useful for verification of production shafts when questions arise regarding geotechnical resistance. However, the capacity limitations of the test device may still be an impediment to the use for large scale tests of high capacity drilled shafts compared to the capacity of the bi-directional test. Rate effects must also be considered. Some of the advantages and limitations are summarized below.

Advantages of rapid load testing include:

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

Limitations of rapid load testing include the following:

- Maximum test load is high, but still limited compared to bi-directional tests
- Rapid loading method; rate effects must be considered
- Cannot assess creep effects from rapid loading
- Mobilization costs for reaction weights

13.2.2.3.4 High Strain Dynamic Load Test

High strain dynamic load testing is a relatively mature technology which has been used for many years with driven pile foundations; the blow of a pile driving hammer on the top of the pile provides the loading for a dynamic load test, if suitable measurements are obtained so that the applied load and pile response can be determined. The measurements are obtained using transducers mounted directly on the pile, and a computer model of the pile response to the blow is calibrated to the measurements using a signal matching technique (e.g., "CAPWAP"). The same technology can be applied to drilled shaft foundations with some considerations for the different nature of a drilled foundation. A description of the application of high strain dynamic tests on drilled shafts is provided by Robinsons et al. (2002).

13.2.2.3.5 Loading Apparatus and Procedure

The procedure for testing deep foundation elements using high strain dynamic load testing is described by ASTM Standard D 4945-00. The high strain dynamic load is imposed by the impact of a falling mass which typically has a weight around 1% to 2% of the desired test load. The impact can be accomplished by either a specially fabricated drop weight test apparatus as shown at left in Figure 13-35, or a large pile driving hammer as shown at right. The hammer shown was used to mobilize axial resistance of approximately 4,000 tons on a 6-ft diameter drilled shaft supporting the column shown in the photo.




Figure 13-35 Dynamic Load Test of Drilled Shafts using Drop Weight (left) and Pile Hammer (right). [Photos courtesy of Pile Dynamics, Inc. (left) and Ross McGillivray (right)]

The high strain dynamic load test setup should always be modeled prior to the test using a wave equation model for specific shaft size and anticipated axial resistance (Hussein et al., 1996). Because the high impact velocity can potentially produce significant compression and tension forces in the shaft, the blow must generally be cushioned using a cushioning material such as plywood. Increasing the drop weight from 1 to 2% of the test load to a value closer to 5% of the test load and using a softer cushion provides another strategy to limit stress in the shaft; this approach results in an impulse of similar duration to the rapid load test described in Section 13.2.2.3.

Measurements of force and velocity at the top of the shaft are needed to perform the analyses necessary for dynamic load testing. These measurements are typically obtained using a Pile Driving Analyzer® (PDA) or similar device. The measurement of force, F_ε is obtained from the strain transducer as:

$$F_\varepsilon = \varepsilon EA \qquad 13\text{-}6$$

where: ε = measured strain
E = elastic modulus of the shaft
A = cross sectional area of the shaft

The measurement of force is also obtained from the acceleration measurement. The acceleration is integrated to obtain particle velocity, and the force proportional to velocity, F_v is obtained by:

$$F_v = \frac{vEA}{c} \qquad 13\text{-}7$$

where: v = measured particle velocity
E = elastic modulus of the shaft
A = cross sectional area of the shaft
c = compression wave speed in the shaft

The determination of the forces applied to the shaft is therefore a function of the elastic modulus, cross sectional area, and compression wave speed of the shaft. The force measurements derived from the two separate types of measurements should be proportional if good measurements are obtained. The measurements from transducers on opposite sides of the shaft should be very similar if the impact is centered and square to the top of the shaft. In cases where the top of the shaft is inaccessible, measurements of force can be made using an extension to the top of the shaft composed of a very heavy wall pipe section which is instrumented and calibrated to measure force. The drop weight can also be instrumented with an accelerometer, and the deceleration of the mass used to measure the force applied to the top of the shaft (F = ma). However, to perform signal matching analyses using the measured response of the drilled shaft itself, it is necessary to obtain measurements directly on the drilled shaft.

To obtain measurements of force and velocity at the top of the shaft, it is necessary that the top of the shaft be exposed so that strain gauges and accelerometers can be mounted onto the concrete or casing, as illustrated in Figure 13-36. Gauges should be mounted at four points 90° apart around the sides of the shaft. Although the ASTM standard requires that the gauges be mounted at least 1.5 shaft diameters below the top of the shaft, one diameter below the top may be sufficient for drilled shafts with four sets of transducers. The placement of gauges on opposite sides is intended to provide detection of an uneven blow and averaging out of small variations due to bending.

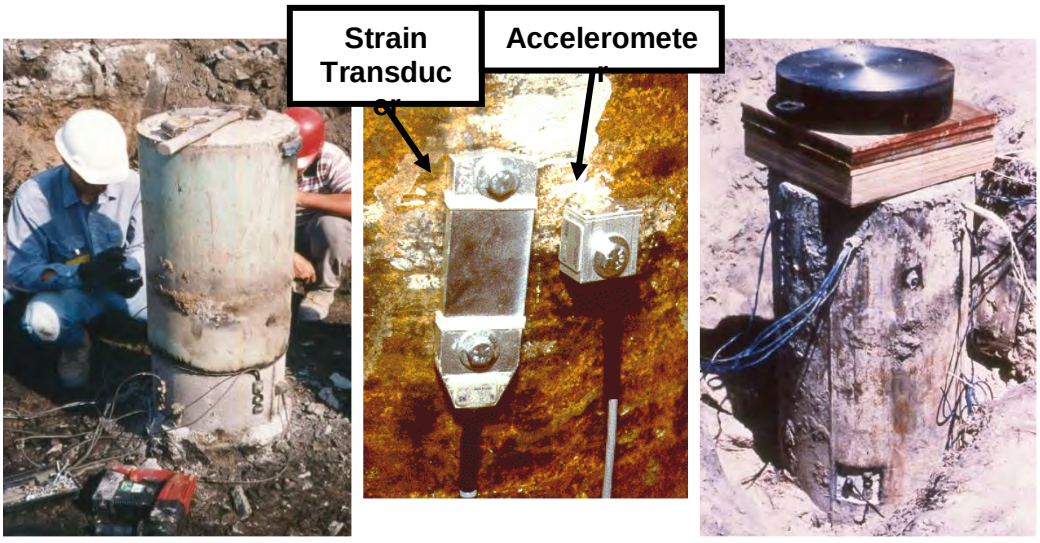


Figure 13-36 Transducers for Measurement of Force and Acceleration (Photos courtesy of Pile Dynamics, Inc.)

Note that the distance required to mount transducers below the shaft top could necessitate a significant excavation for shafts of large diameter. Because of this fact, and also because there is often reinforcement extending above the top of the shaft, one common practice is to cast a concrete segment above the top of shaft for the application of the impact blow. This separate "drive head" casting can be removed with jackhammers after completion of the dynamic testing. The use of a drive head casting has several benefits:

- It allows the transducers to be mounted on a segment of known and uniform cross section.
- Since it can easily be cast in the dry, the segment cast above grade should have concrete of high and uniform quality.
- The need for excavation is avoided.
- Any reinforcement extending above the top of the shaft can be incorporated into the drive head.

Gauges may sometimes be placed on the permanent casing near the top of the shaft. However, the casing and shaft must act as a composite section without slippage; otherwise, the measurements on the casing would not accurately reflect the shaft response. A length of more than one shaft diameter may be necessary to develop the bond needed for composite action. To ensure that the measurements more accurately reflect the shaft response, it may be necessary to cut a window in the casing and mount gauges directly on the concrete, as illustrated on the photo at right in Figure 13-36.

The displacement as a function of time is obtained from double integration of acceleration, and the permanent set after several blows is checked using surveying instruments or by reference to a known datum.

Two to ten impact blows are typically obtained to mobilize the axial resistance of the shaft. The first blow is often not the most useful since there may be insufficient permanent displacement to mobilize the base resistance of the shaft.

13.2.2.3.6 Derivation of Static Axial Resistance

The fundamental concepts used to evaluate high strain dynamic testing involve the propagation of a compression wave in a one dimensional rod. This problem is the basic problem of a hammer blow on a driven pile foundation. The basics of wave mechanics, energy transfer, and analyses of pile capacity from dynamic load testing is described thoroughly in the FHWA Driven Pile Manual (Hannigan et al., 2016). This section provides an overview of the process and description of those issues specific to drilled shafts.

A device such as the PDA or similar system typically includes closed form algorithms for quickly computing axial resistance from the measurements of a blow. The FHWA Driven Pile Manual describes the "Case Method" with different variations of the equations used depending upon the nature and distribution of the soil resistance acting on the pile. However, a more rigorous method is recommended for analysis of dynamic load tests on a drilled shaft.

The derivation of axial static resistance from a high strain dynamic test is typically obtained using a computer analysis that involves signal matching as illustrated in Figure 13-37. The basic idea is that a model of the shaft with soil resistance is constructed and used to compute the response at the location of the transducers due to the propagation of the input force and velocity from the blow. The computed response is compared with the measurements, and the input parameters (distribution of soil resistance, soil damping, etc.) are modified until the computed and measured responses are in agreement.

The computational model for performing the signal matching includes a numerical model of the shaft as a series of individual segments, each having mass and stiffness and each attached to a soil model as illustrated in Figure 13-38. The soil includes components of static and dynamic resistance, and the entire numerical model is used to perform a time-domain analysis of the propagation of the input force and velocity pulse from the blow. Reflected waves from each individual segment affect the computed response at the shaft head (location of transducers) and provide a basis for the signal matching process. FHWA-sponsored research lead to the development of the computer code CAPWAP (CAse Pile Wave Analysis Program). This code is now available as a proprietary program from Pile Dynamics, Inc. and is the most widely used software product in North America, although other similar software products are also now available for this purpose.

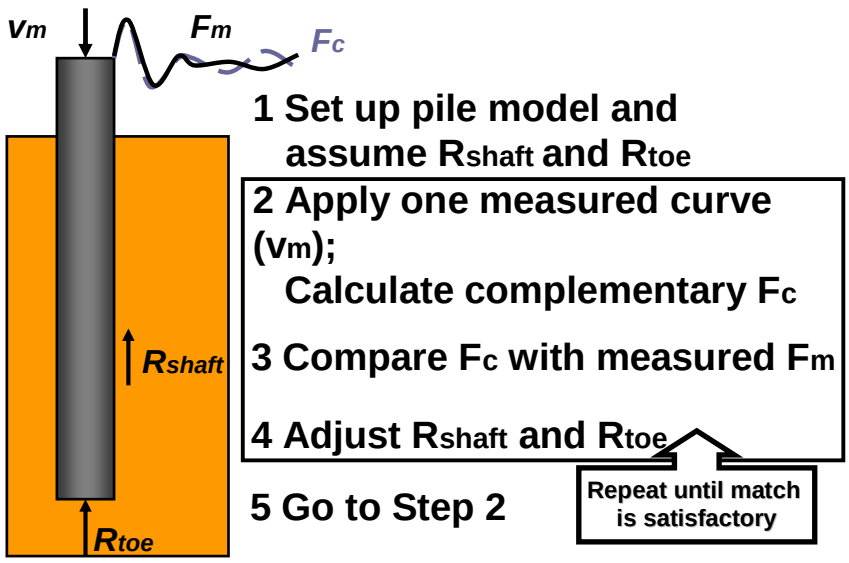


Figure 13-37 Signal Matching Concept

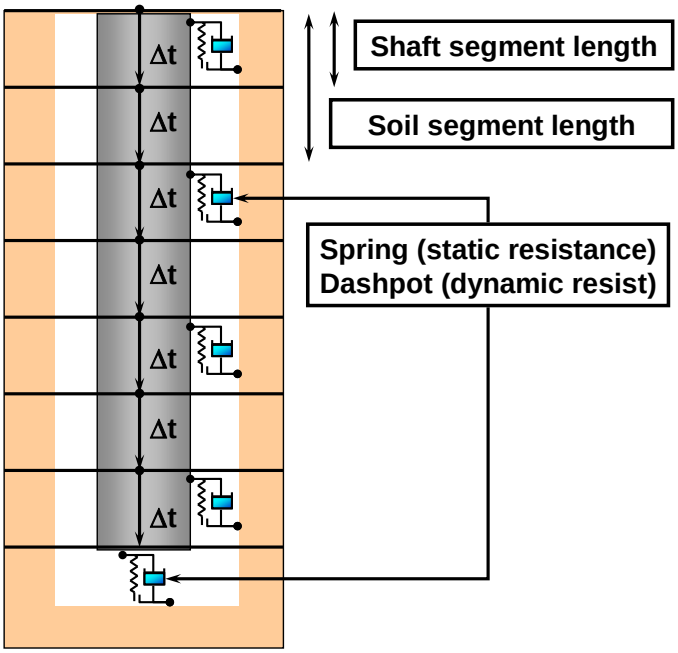


Figure 13-38 Computational Model of Shaft/Soil System

Once a good match of the computed and measured response is achieved, the distribution of side and base resistance in the model is considered as a good estimate of the behavior of the test shaft. While the solution obtained from the signal matching is not unique and should not be taken as a precise measurement of the distribution of resistance along the shaft, the computed response is sufficiently constrained that the model is considered to be a reasonably reliable indication of the approximate distribution of resistance. An example of the measured and computed forces at the top of a pile is illustrated on Figure 13-39, with the time scale marked in terms of L/C, length of pile divided by compression wave velocity. A reflection from

the pile toe would occur at a time equal to 2L/C. This figure illustrates the influence of the various components of shaft and base (toe) resistance on the measured and computed forces at the top of the shaft.

The damping component of resistance is important, because the total soil resistance to penetration is the static resistance plus the damping resistance. The analysis is most sensitive to total resistance, and if damping were underestimated, then the static resistance might be overestimated (and vice versa). Therefore, it is essential that the soil conditions be known from borings and drilled shaft logs so that the damping values determined from the analysis be checked for consistency with established guidelines.

An additional and very important factor which influences the reflections at the top of a drilled shaft is the variations in shaft impedance (EA/c) along the length of the shaft. A change in cross sectional area or in modulus of the shaft as illustrated in Figure 13-40 will produce reflections at the shaft top. Since variations in shaft impedance complicate the analysis, it is important that the soil resistance and the physical profile of the shaft be both considered interactively during the signal matching exercise and compared with the known soil profile. One method for estimating the impedance profile of the shaft is to perform low-strain integrity testing with the sonic echo technique on the shaft, as illustrated in the photo at right in Figure 13-40. The sonic echo test is described in Chapter 16 of this manual.

Another issue related to high strain dynamic testing of drilled shaft foundations is the relatively large displacement which may be required to mobilize the end bearing compared to the displacement achieved on a single blow. A good high strain dynamic measurement generally requires that the permanent set on a single blow should not exceed about 1/3 inch (e.g., 3 blows or more per inch); larger set tends to induce tension waves in the shaft which could be damaging and also affect the interpretation of the measurements. For a 4 to 7-ft diameter shaft, this magnitude of displacement only represents about 0.7% to 0.4% of the shaft diameter. To achieve a displacement in the 3% to 5% range for 4 to 7-ft diameter drilled shafts requires a movement in the range of 1.5 to 4 inches, and thus 5 to 12 blows would be required if the impact were achieving the upper limit of 1/3 inch per blow. Therefore, the first few blows may not provide a good indication of the base resistance of the shaft. After too many blows, the side shear may tend to be reduced because of the dynamic effects (the shaft is being driven like a pile), and therefore many blows on the shaft may not provide a good indication of the static resistance in side shear. The optimum blow to use as an indication of static resistance is often the one which provides the greatest magnitude of derived static resistance.

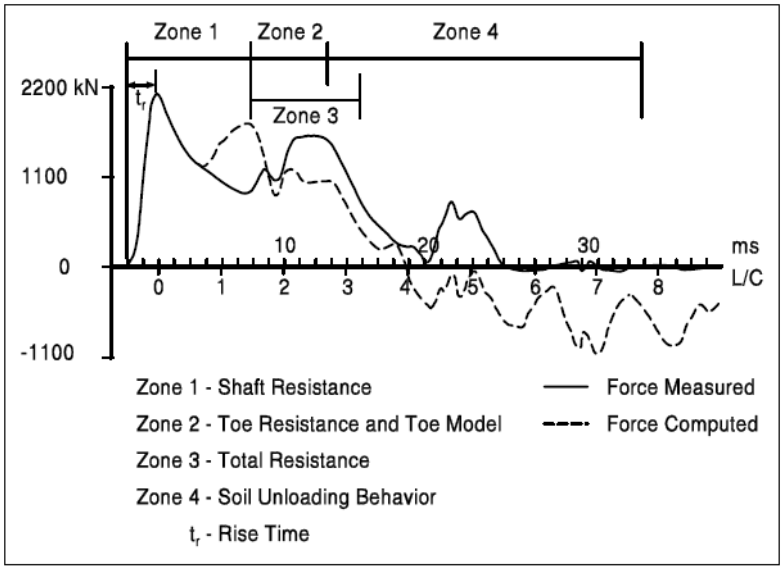


Figure 13-39 Factors Most Influencing CAPWAP Force Matching (after Hannigan, 1990)

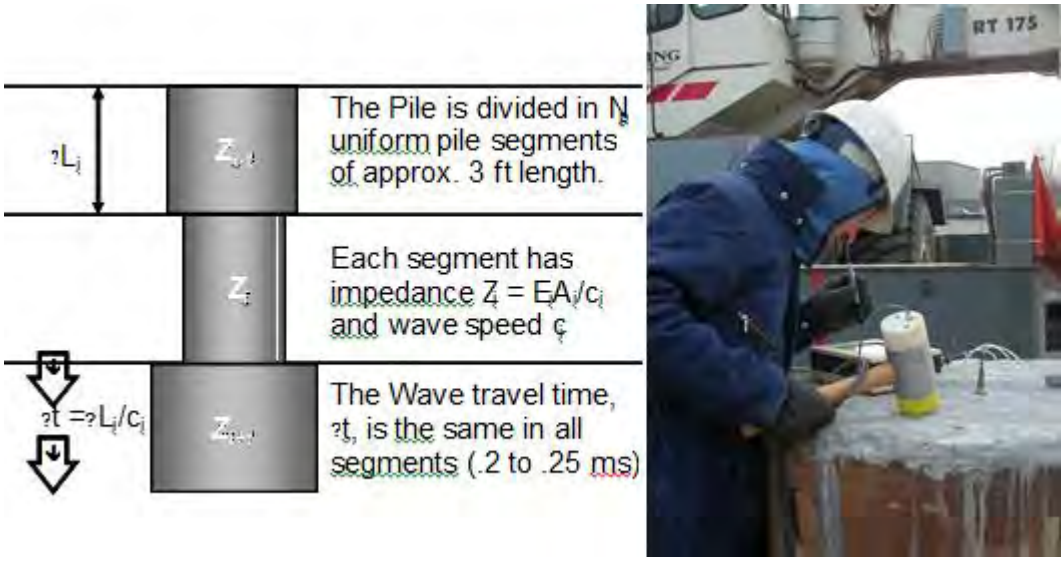


Figure 13-40 Impedance Profile of the Drilled Shaft

A discussion of this effect and other factors affecting the interpretation of dynamic load tests on drilled shafts and large diameter piles is presented by Rausche et al. (2008), who recommend performing signal matching analyses on several successive blows in order to interpret the results appropriately. The data presented in Figure 13-41 illustrate the computed static response (matched to dynamic measurements) of a 6-ft diameter by 65-ft long drilled shaft subjected to four successive blows. The four blows produced a total cumulative net displacement of just over ½ inch (about 14 mm). The first two blows only partially activated the end bearing while the second blow activated the maximum side shear. The third and fourth blows engaged progressively more end bearing, but the magnitude of the side shear (the difference between the top and the toe load) begins to diminish after the second blow.

Note that signal matching analyses of a dynamic test with insufficient force to mobilize the shaft capacity will only indicate a resistance equal to that mobilized during the test.

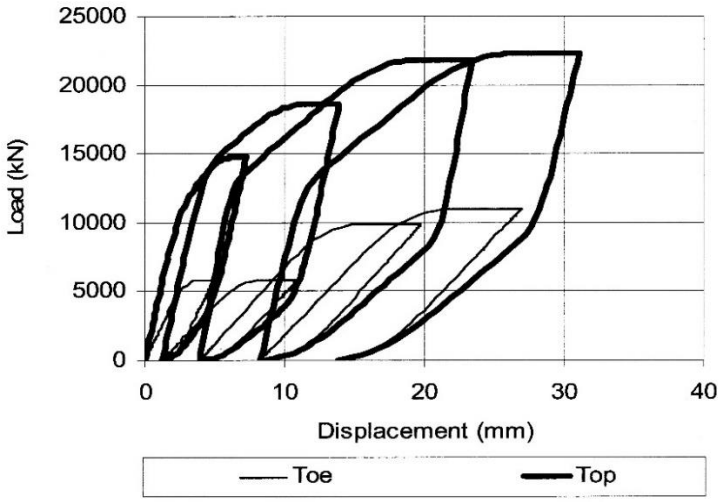


Figure 13-41 Calculated Load Cycles from Signal Matching Analyses for Four Blows on a 6-ft Diameter Shaft; 1 kip = 4.45 kN; 1 inch = 25.4 mm (after Rausche et al, 2008)

The AASHTO code does not address design of drilled shafts based on high strain dynamic load tests, but provides for the use of a resistance factor of 0.65 for driven piles based on dynamic load tests. However, this approach is for piles which are to be driven to a consistent driving resistance with the same hammer used to perform the dynamic load test, and with at least one dynamic load test pile per pier. The consistency of the axial resistance of drilled foundations must be verified by other means during inspection of the installation process. Note that AASHTO Section 10.5.5.2.4 provides for a maximum resistance factor of 0.70 for drilled shafts calibrated to site-specific static load tests, with a 20% reduction in resistance factor for non-redundant shaft foundations such as a single shaft supporting a single column.

Considering that some additional variability is introduced by performing a high strain dynamic load test compared to a conventional static load test, it is preliminarily recommended that general design of drilled shaft foundations calibrated to site-specific high strain dynamic load test data should utilize a resistance factor which is 0.10 less than the factors outlined in AASHTO Section 10.5.5.2.4 for design based on static load tests. Therefore, a maximum resistance factor of 0.60 is indicated for design of drilled shafts when high strain dynamic load tests are calibrated to site-specific static load tests, with lesser values used for non-calibrated high strain dynamic load tests, non-redundant foundations or unusually high site variability. These recommendations are considered to be conservative considering that high strain dynamic tests likely do not mobilize the full end bearing resistance of the tested shaft.

13.2.2.3.7 Advantages and Limitations of High Strain Dynamic Load Testing

Most of the advantages and limitations of high strain dynamic load testing are apparent from the previous discussion. Loads of up to 4,000 tons (or potentially more with a very large drop hammer) can be applied to the top of a drilled shaft foundation without the need for a reaction system and with only modest preparations of the shaft top during construction. These capabilities make dynamic load testing a cost effective tool which is useful for proof tests on production foundations or for performing multiple load tests at a site. However, the capacity limitations are still an impediment to the use for large scale tests of high capacity drilled shafts compared to the capacity of the bi-directional test. Rate effects due to damping resistance represent an additional variable. Some of the advantages and limitations are summarized below.

Advantages of high strain dynamic load tests include:

- Large test load, applied at top of shaft
- Can test existing or production shaft
- Economies of scale for multiple tests
- Low cost compared to other forms of testing
- Amenable to verification testing on production shafts
- Reaction system not needed

Limitations of high strain dynamic load tests include the following:

- Capacity high, but still limited compared to bi-directional tests
- Test includes dynamic effects which must be considered by a signal matching analysis
- The interpretation of measurements on the shaft is affected by the shaft modulus, area, and uniformity in the top 1 to 1.5 diameters.
- Test must be designed to avoid potential damage to the shaft from driving stresses
- Cannot assess creep effects from dynamic loading
- Mobilization costs for a large pile driving hammer or drop hammer

- The shaft shape must be considered in the analysis; the shape might be known from installation records, or derived from the signal matching process guided by the soil profile
- Shaft displacement likely insufficient to fully mobilize the base resistance. If a loss of side resistance is observed after multiple blows, then this issue complicates the interpretation of the results

13.2.2.3.8 Uplifting Test

For projects where drilled shafts are to be subjected to substantial uplift loading (e.g., because of overturning moments applied to the structure through seismic events or extreme winds, or foundations at the anchorage end of permanent cantilevers), uplift tests should be considered. The decision to perform uplift tests must consider many of the considerations previously discussed for compression load tests. Further, uplift tests may not be needed for cases where drilled shaft penetration and available side resistance is predominantly governed by compression loads on the shaft,

Standards and procedures for performing static axial tensile load tests for deep foundations are included in ASTM test designation D 3689-07. An arrangement for the performance of a conventional uplift test of a drilled shaft is shown in Figure 13-42 (Sacre, 1977). The key feature of the arrangement is that some of the longitudinal rebars that are embedded full length in the test shaft, extend upward to a point well above the head of the test shaft. These extended rebars may be made of high-strength steel and are often equipped with a threaded connection. The reaction beams may be supported on compression shafts or even on surface mats if the ground conditions are favorable. Other than the position of the loading jack and reaction beams, the test is similar to a conventional compression test. Other types of uplift tests are possible (e.g., Johnston et al., 1980), and bi-directional load tests can be used to determine resistance to upward directed load.

A couple of special considerations are noteworthy with respect to uplift loading as compared to conventional compression loading: 1) the reaction system may need to be relatively far away in some cases to avoid affecting the results, and 2) the shaft may be in tension (load applied at top of shaft) or compression (load applied at shaft base) depending upon how the test is arranged.

In the case of a shaft anchored into rock which may contain fractures or seams, the zone of a possible “cone breakout” near the top of the rock formation may be of significant size. The reaction foundations must be located sufficiently far away from the uplift test shaft so that they do not influence the rock mass affecting the uplift resistance, as illustrated in Figure 13-43. This effect is particularly important for reaction foundations bearing on surface mats. The necessary lateral distance to reaction foundations may be influenced by the embedded length of shaft, depth to rock, and other factors. In some cases it may be desirable to utilize reaction shafts to avoid surface stresses in the zone of influence of the test shaft.

A production drilled shaft which is loaded in uplift typically has the load applied to the shaft by pulling the reinforcement at the top of the shaft, thus placing the foundation into tension. With some types of anchorage foundations, it is possible to sleeve the reinforcement through the length of the shaft to an anchorage plate at the base (or use a bi-directional loading system at the base), thus placing the shaft into compression for an uplift load. The geotechnical response in side shear for each of these two conditions may be different, particularly for a shaft which is anchored into rock as illustrated in Figure 13-44. The Poisson’s ratio effect due to elastic deformation of the shaft for the tension case will tend to reduce the lateral stress at the shaft/rock interface, whereas the opposite is the case for the shaft loaded in compression. The best approach to uplift loading is that the test should replicate the production condition, if possible.

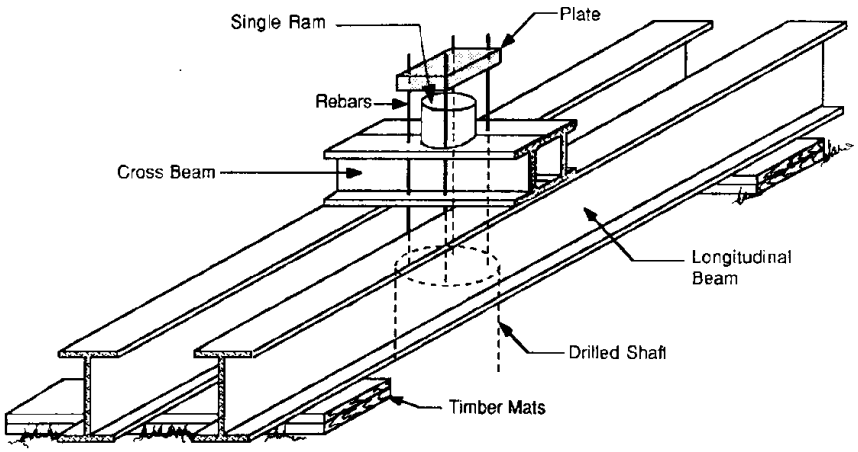


Figure 13-42 Arrangement for Testing a Drilled Shaft under Uplift Loading

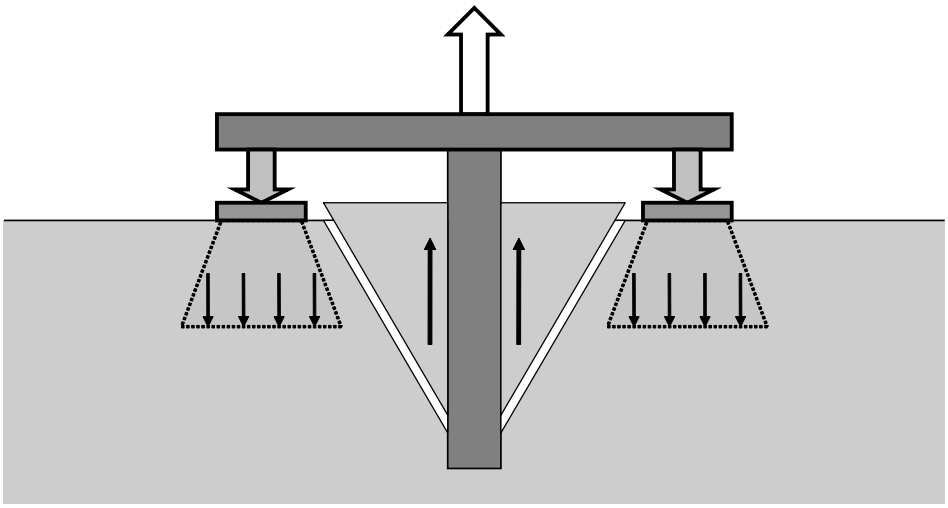


Figure 13-43 Location of Reaction Mats Relative to Uplift Shaft in Rock

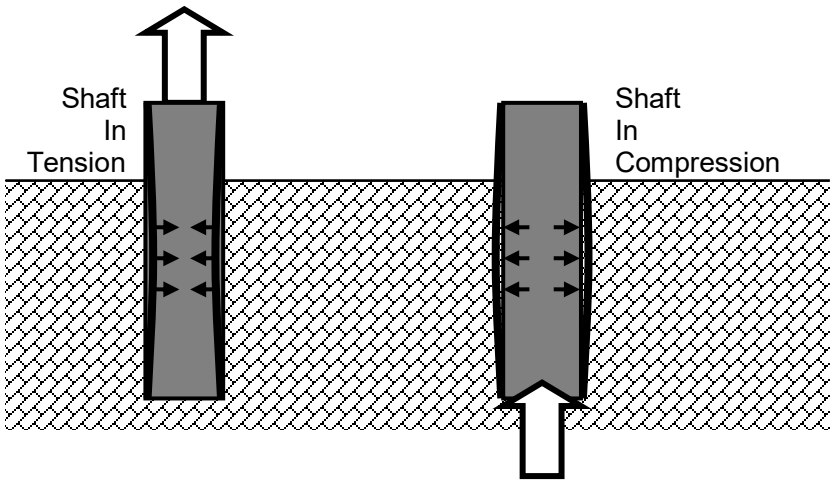


Figure 13-44 Uplift Shafts in Tension versus Compression

13.2.3 Instrumentation

Instrumentation is a critical part of the load test system to obtain measurements of drilled shaft performance during the test. The instrumentation system must be robust and reliable as well as accurate. In addition to the measurement of load and displacement at the top of the shaft, measurements of strain along the length of the shaft can provide important information about the distribution of load transfer in side shear and end bearing. This section provides an overview of the instrumentation used to measure loads and displacements on and within the test shaft.

13.2.3.1 Application and Measurement of Load

Measurement of load is obtained using hydraulic pressure measurements on the jack and using electronic load cells. With top-down static tests, it is desirable to obtain both measurements to provide redundancy. In general, the use of a calibrated electronic load cell provides the more accurate measurement of load, as this device is not subject to stiction (static friction). Both can be subject to measurement errors due to misalignment and eccentricity in the loading system. The rapid load test should include a calibrated electronic load cell within the loading apparatus. The bi-directional embedded jack relies upon a calibration of the jack load to measured pressure. High strain dynamic tests do not have a direct measure of applied load except as inferred from strain and acceleration measurements on the shaft.

In any instrumentation plan, it is desirable to provide redundancy in the measurement of load. Even if a load cell is used as the primary means of determination of load, measurement of jack pressure provides a backup system. If only jack pressure is measured, two separate and independent measurements of pressure should be included.

13.2.3.1.1 Hydraulie Jack

Hydraulic jacks are used to apply static load to the shaft via applied hydraulic pressure that is applied with a pump. The capacity of the jack is a function of the size of the ram and the magnitude of pressure which can be used in the jack and applied by the pump. Electric or hand-operated pumps may be used, depending on the size of the jack and volume of fluid required to operate the system. For large loads, multiple jacks may be used in parallel, as illustrated in the photo at right in Figure 13-45. The magnitude of travel in the jack is another potential consideration, particularly where large shaft displacements are anticipated. Note that the bi-directional load cells shown in Figure 13-19 are also simple hydraulic jacks which are designed to be embedded in the shaft.

The jack should be calibrated so that the applied pressure corresponds to a known load during the test. Pressure may be observed using an analog pressure gauge on the jack or with in-line pressure transducers which may be recorded with a data logger. The most reliable location for the pressure sensor is near the jack rather than at the pump to reflect losses in the lines between the sensor and the jack.

Besides the measurement of pressure, the major source of error in using a jack as an indicator of load is the stiction in the jack itself, which may vary with rate of load and with the direction of movement (up or down) of the jack. If any significant misalignment is present in the loading system, the jack may be loaded eccentrically, increasing the magnitude of stiction. Besides the obvious potential to damage the jack, eccentricity in the applied load affects the relation between the measured pressure and the load applied to the test shaft by the jack. The reaction beam must not be subject to twisting as the load is applied because any out-of-plane distortion produces eccentricity and misalignment in the loading system. A hemispherical bearing or swivel head system is typically used in the jack system to accommodate small angular differences between the reaction beam/plates and the end of the ram.

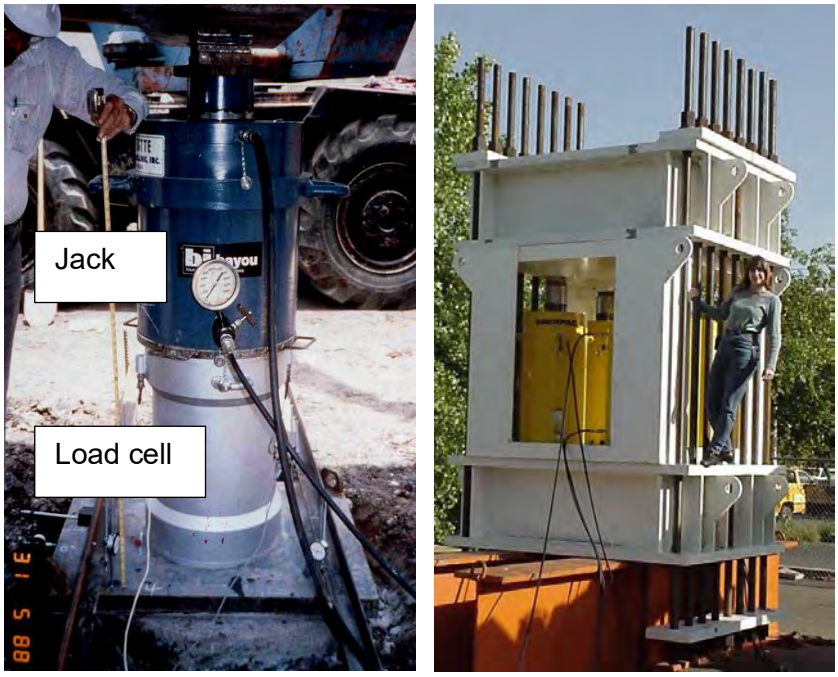


Figure 13-45 Hydraulic Jack with Load Cell (left) and Multiple Jacks in Parallel (right)

13.2.3.3.2 Load Cell

A load cell is a passive element included within the loading system for the purpose of measurement of load. A typical load cell is illustrated on the left of Figure 13-45. This cell is an elastic steel element with strain gauges that have been calibrated to load. Another load cell was illustrated in Figure 13-30 as a part of the rapid load test apparatus. In general, load cells are considered to provide a more reliable measure of load than a hydraulic jack because the passive cell is not subject to stiction as is the hydraulic ram. However, misalignment and eccentricity in applied load can still result in significant measurement errors, even with a load cell. The types of strain gauges used in load cells may vary and the requirements are dependent upon the rate of loading; strain gauges are described in Section 13.2.3.5.

13.2.3.4 Measurement of Shaft Displacement

Determination of shaft displacement at the shaft top or point of load application should also include redundant measurements.

13.2.3.4.1 Shaft Top

The most widely used method of measuring shaft top displacement for conventional static load tests is to mount a series of dial gauges or displacement transducers onto a reference system. Displacement gauges should ideally be mounted at several symmetric points atop the test shaft so that an average value can be obtained. Analog dial gauges may be read and recorded manually, and other types of displacement transducers such as LVDT's (linear variable differential transformers) or linear potentiometers (essentially a variable resistor) may be monitored with an automated data logger. Sufficient accuracy of measurements (generally 0.01 inches is adequate) can be readily obtained with a variety of instruments. However, analog dial gauges may be somewhat complicated and easily misread; consider the gauges illustrated in Figure 13-46.

The greatest source of error is likely to be related to the stability of the reference system, which must be supported well away from the test shaft and any reaction foundations. The reference system should also be free from temperature distortions and wind or waves. The system illustrated in Figure 13-9 was supported in a direction orthogonal to the reaction system and constructed of timber, which is less prone to temperature fluctuations. Shading the reference system is also advisable on a sunny day. When working over water, it may be necessary to support the reference system on a pile foundation which is isolated from wave action by an isolation casing.



Figure 13-46 Dial Gauges: What Do You See? (answer: 0.623, 0.797, 0.804)

A backup system for displacement measurement may consist of a piano wire with a scale and mirror and/or optical measurements using a level. The scale is attached to the mirror which is mounted onto the test shaft, and allows the observer to make a consistent reading of the scale relative to the wire by aligning the image of the wire with the reflection of the wire in the mirror. Optical measurements using a surveyor's level may be used as a backup and to observe movements of the reaction shafts or piles, but the surveyor's level generally does not provide sufficient precision for use as the primary measurement system.

Recent developments include optical methods as illustrated on Figure 13-47. This system is now commonly used as the primary displacement measurement because the optical scale can be monitored electronically from a position that is far away from the test location, and data logged with other electronic measurements.

Double integration of acceleration measurements are used to provide the primary means of displacement measurement during both the high strain dynamic and rapid load tests. The permanent set after loading is checked after each blow or loading using a measurement of marks on the test shaft against a reference system such as a piano wire or optical scale.

13.2.3.4.2 Below Grade

Telltales may be used to obtain measurements of displacement below the top of the shaft. Telltales consist of unstrained rods extending through vertical tubes to a mounting position below grade, as illustrated in Figure 13-48. Dial gauges or electronic displacement transducers can then be mounted on each rod to measure the displacement at various points below the top of shaft, as illustrated in Figure 13-47. Telltales are often used with bi-directional load tests to measure shaft movements above and below the load cell(s).

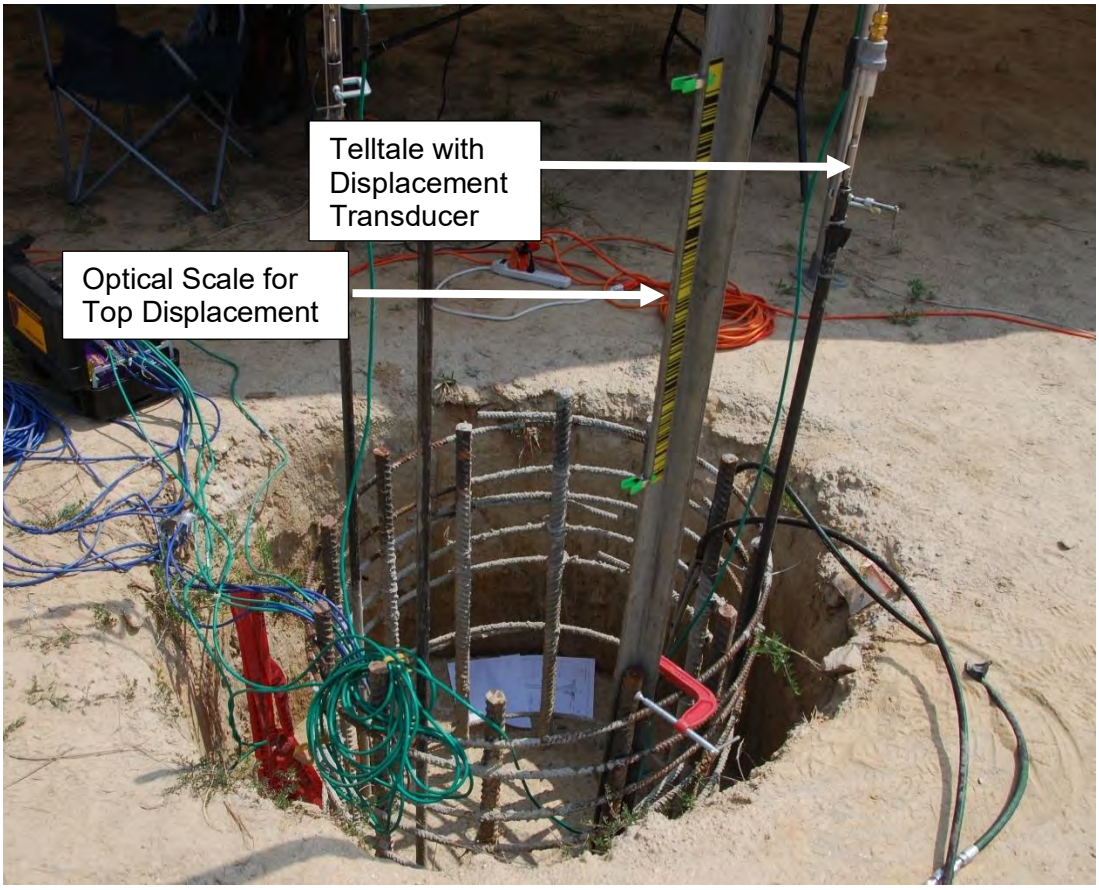


Figure 13-47 Measurement of Displacement on the Test Shaft

13.2.3.4.3 Measurement of Strain

Strain measurements are a key component of axial load tests to determine the distribution of load transfer along the test shaft. Several types of strain gauges may be used, as described in the following sections. The reliability of the measurement of strain may be excellent, but the interpretation of load transfer from the measurements is less reliable because of the physical characteristics of the drilled shaft. A discussion of the interpretation of strain data follows the discussion of strain measurements.

Most strain gauges used in drilled shaft applications are mounted on "sister bars" as illustrated in Figure 13-49 and Figure 13-50. The sister bar typically consists of a #4 reinforcing bar about 4-ft long with the gauge mounted near the middle. The bar is intended to bond to the concrete via the development on either side of the gauge so that the measurement of strain on the bar reflects the average strain in the drilled shaft at that location.

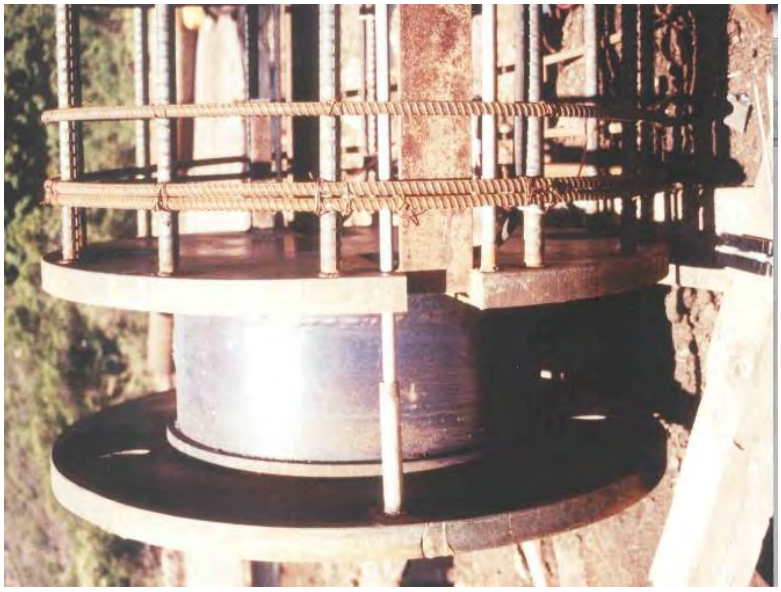


Figure 13-48 Telltale Rod for Measurement of Displacement Below the Top of the Test Shaft

13.2.3.4.4 Resistance-type Strain Gauges

Resistance-type strain gauges are composed of a "foil" gauge that is bonded to the sister bar, and which has the properties of varying resistance as the gauge is strained in a specific direction. The electronics and technology associated with resistance gauges has markedly improved in the last 10 years, with gauges that operate at lower voltage and with very high reliability. The gauge illustrated at left in Figure 13-51 may typically be 1/4 to 1/2 inch in length, has lead wires soldered to the connector tabs at the bottom, and is sealed for waterproofing after bonding to the sister bar. Gauges are arranged to form a circuit called a "Wheatstone" bridge as illustrated at right in Figure 13-51. In a typical "quarter bridge" application, the active strain gauge makes up one component of the bridge and the other three "dummy" gauges complete the circuit at the location of the data logger. As the resistance of the active gauge changes, a voltage is measured at the location in the circuit as shown, and this variation in voltage is calibrated to the strain at the gauge location for a known applied voltage level. It is possible to use multiple active gauges on a single sister bar to form a "half bridge" or even a "full bridge" circuit; use of multiple active gauges can increase the resolution of the measurement, although the quarter bridge is usually of sufficient precision for drilled shaft load testing. A precision of +/- 1 microstrain (10^{-6} inch/inch) is typical with a quarter bridge circuit.

Resistance type strain gauges are inexpensive and reliable if water-proofed properly, but their greatest advantage is perhaps the fact that these gauges can be sampled thousands of times per second. Therefore, resistance gauges are ideal for dynamic measurements. The externally mounted strain sensor shown in Figure 13-36 also uses resistance type gauges.

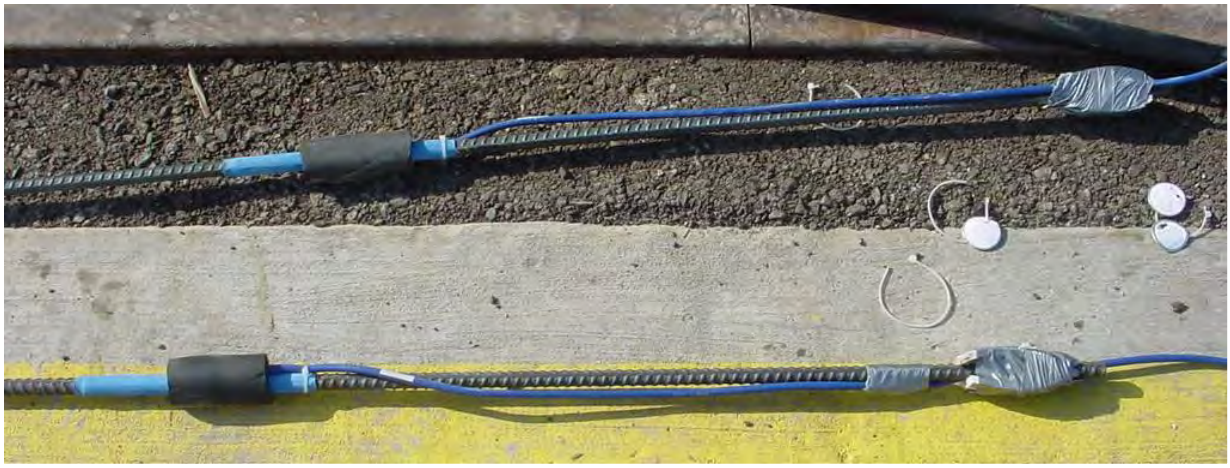


Figure 13-49 Sister-bar Mounted Strain Gauges



Figure 13-50 Sister-bar Mounted Strain Gauges Placed in Drilled Shaft Reinforcement

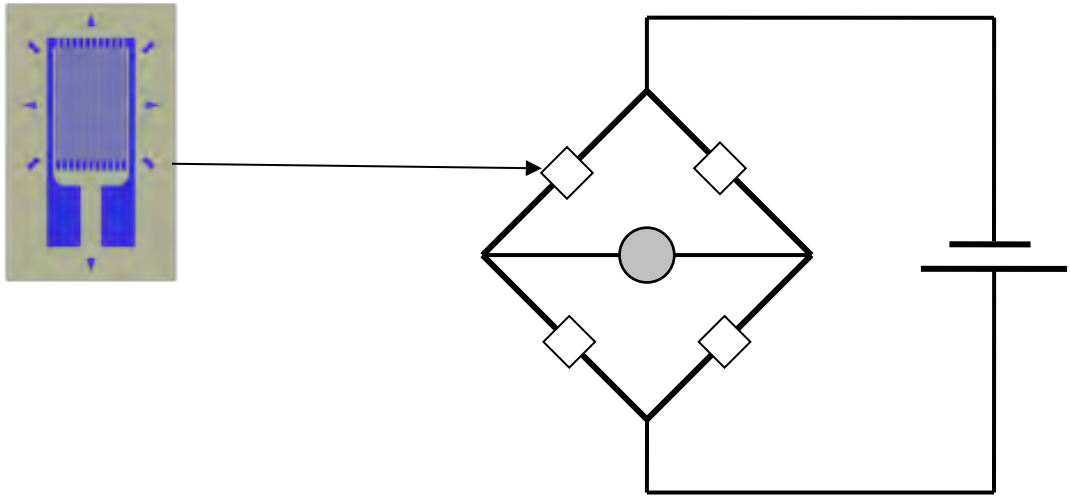


Figure 13-51 Resistance Type Strain Gauge

13.2.3.4.5 Vibrating Wire Strain Gauges

Vibrating wire gauges provide a measurement of strain by correlation with the harmonic frequency of a wire which makes up the gauge. These instruments are mounted on a sister bar for installation in a drilled shaft in a manner similar to resistance gauges and as illustrated on Figure 13-52. The wire within the gauge is stressed to some predetermined level, much the same as a guitar string, and a change in length of the gauge will result in a change in the "pitch" or harmonic frequency when the gauge is "plucked" by an electronic device. The vibrating wire gauge has a specific range of strain over which it can measure, and the pre-set frequency can be adjusted within this range at the time of manufacture in order to provide a greater range in compression (which will reduce the tension in the wire) or tension (which will increase wire tension). The harmonic frequency of the wire and the calibration to strain is affected by temperature, and therefore these gauges must always include a thermocouple for measurement of temperature at the time the strain is measured.

The data logging equipment for vibrating wire sensors is different than would be used for resistance type gauges, for obvious reasons. Each measurement of each gauge involves an electrical instruction to "pluck" the gauge and a short period of time (less than a second) for the system to monitor and measure the frequency of vibration of the gauge. Although the strain measurements can be obtained very quickly relative to the requirements of a static test, the response time required for these gauges precludes their use for rapid or dynamic load tests.

13.2.3.4.6 Fiber Optic Strain Sensors

Fiber-optic tubes have been successfully embedded in bridge decks and other concrete structures to measure the distribution of strain. Gratings can be inscribed at various points on a fiber-optic tube, as illustrated in Figure 13-53 and these reflect coherent laser light at varying wave lengths depending on the strain at the location of the grating. Fiber-optic gauges can also be mounted on sister bars like conventional gauges, and this method extends the effective length of the gauge beyond the small grating spacing. Although fiber-optic strain sensors are less commonly used at present, they are suited to the measurement of strain (and therefore load) distributions in drilled shafts during load tests, particularly tests conducted over long durations, since they can be made to be very stable. They are adaptable to multiplexing of data to the data acquisition unit, and many sensing locations can be established within the shaft at a fraction of the cost of other types of strain sensors. The disadvantages are that expert interpretation is still required and that the data acquisition equipment, although reusable, is initially expensive. Fiber-optic sensing may be considered for long-term monitoring of load transfer patterns in drilled shafts that are in service. For the reader interested in pursuing this type of instrumentation, an excellent overview of fiber-optic sensors for concrete structures is given by Merzbacher et al. (1996).



Figure 13-52 Vibrating Wire Type Strain Gauge

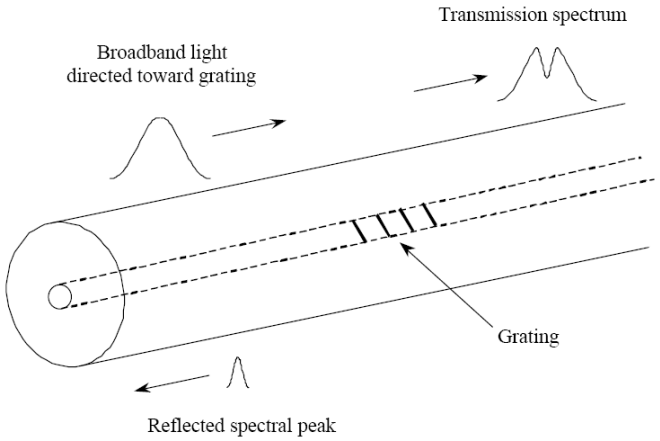


Figure 13-53 Fiber Optic Strain Gauge with Bragg Grating (Schultz, et al., 2001)

13.2.3.4.7 Extensometers

Extensometers can be used to measure strain by obtaining a measurement of displacement between two points along the length of the shaft. The gauge length is thus the distance between the two points and the average strain is the relative displacement divided by the gauge length. Telltales could be used for this purpose, but the displacement measurements are usually too crude and the gauge length too long to provide measurement of strain which is useful for determination of load transfer. However, LVDT's can be anchored at multiple points along a borehole or tube within the drilled shaft to provide accurate measurements of relative displacement. These devices can be recoverable and thus offer economy for multiple use applications relative to embedment gauges. These types of instruments have a history of use in the mining industry and are useful for other geotechnical applications such as settlement monitoring of embankments. However, the strain measurement in a drilled shaft is generally less precise relative to embedment gauges, and the instruments are still relatively expensive.

13.2.3.4.8 Interpretation of Strain Data

The purpose of the strain measurement is to obtain a measure of the axial force in the shaft at the point of each measurement so that the load transfer to the soil between measurement points can be determined.

The reliable measurement of strain at a point along the length of the shaft is best achieved by averaging the measurements of two or more embedment gauges at a given elevation. If there is eccentricity in load or bending stress in the shaft, the strain will vary across the cross section of the shaft. Averaging two or four separate gauges can eliminate the effects of bending. In addition, the use of multiple gauges provides redundancy in the case of a damaged or inoperable gauge.

For a given strain measurement, the force in the shaft is determined according to Equation 13-8:

$$F = \varepsilon(AE) \qquad 13\text{-}8$$

where: F = axial force in the shaft
ε = measured average axial strain
A = area of cross section
E = modulus of elasticity of drilled shaft

Even if the strain measurement is extremely precise, the determination of the axial force is affected by the area of the cross section and the modulus of elasticity. The actual area of the shaft is not usually known with a high degree of accuracy, but the knowledge of the cross sectional area can be much improved with the use of a borehole caliper as described in Section 13.2.1.5. The actual modulus of elasticity of the concrete at the time of testing is dependent upon the compressive strength and the age of the concrete. Since a drilled shaft typically includes both concrete and steel reinforcement, the AE represents a steel/concrete composite section. In short, a cast-in-place reinforced concrete drilled shaft does not make a very accurate load cell, so the interpretation of the strain measurements to determine axial force must be tempered with judgment regarding the precision of the force determination.

Once the force is determined at the elevation of each strain measurement, a force vs. length plot can be developed, as illustrated in Figure 13-54. The top most point is the measurement from a load cell or calibrated jack, the remaining points below grade are interpreted from strain measurements. Each curve represents a measurement for a specific load atop the shaft at the top displacement shown in the legend.

The difference between the force at each measurement elevation represents the load transferred to the soil or rock over the length of that segment. For instance, for the measurements corresponding to 1 inch of top displacement, the load at elevation 150 ft was determined to be 1,164 kips and the load at elevation 140 ft was determined to be 912 kips. Therefore, the load transferred to the soil in side resistance over this segment is 1164 - 912 = 252 kips.

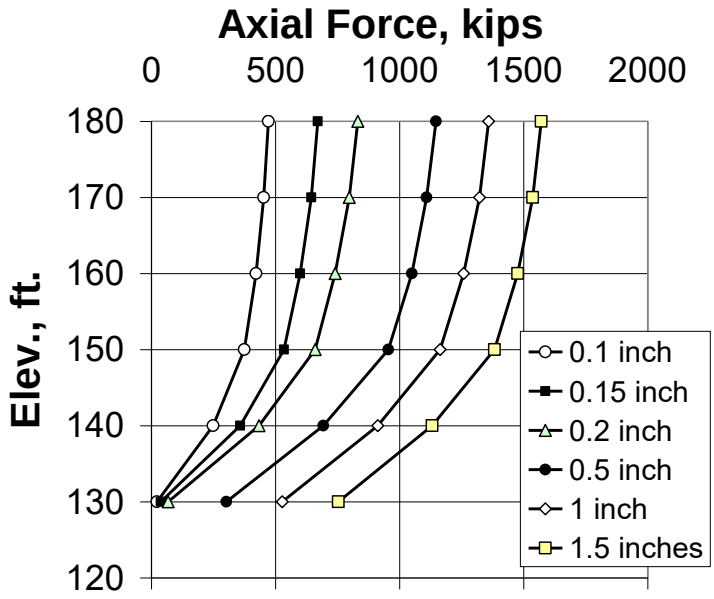


Figure 13-54 Example Plot of Force versus Elevation for a Top-Down Load Test

The unit load transfer is determined by the difference in load between two points comprising a segment of the test shaft divided by the surface area of that segment of the test shaft. For instance, if the test shaft in Figure 13-56 had a 4-ft nominal diameter between elevation 150 ft and 140 ft, then the surface area of this segment is $4 \cdot \pi \cdot 10 = 126$ ft^2, and the unit load transfer for the measurement at 1 inch of top displacement was 252k/126 ft^2 = 2.0 ksf. The average displacement at a given segment location can be determined from the displacement at a measured location less the elastic shortening (average strain times length) from the measurement location to the segment. In this manner, unit load transfer versus displacement can be derived as illustrated on Figure 13-55.

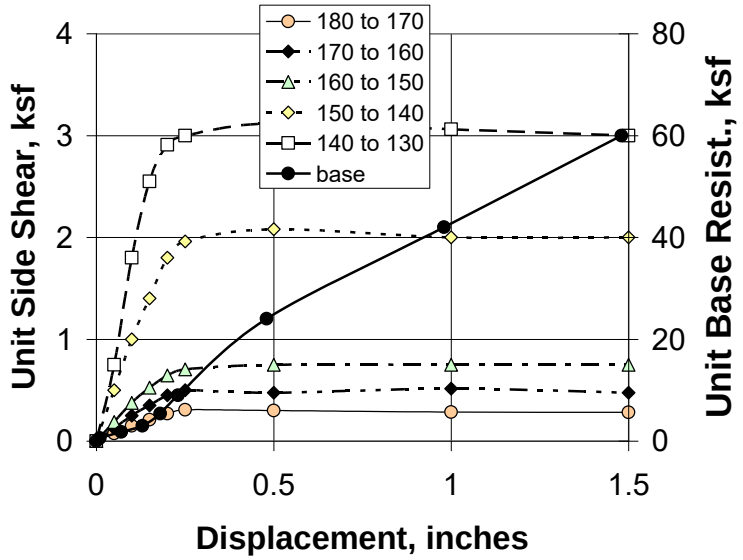


Figure 13-55 Example Plot of Unit Load Transfer Curves

A real example is illustrated in Figure 13-56. A schematic of the test setup is illustrated at top left. This shaft was 4-ft diameter and approximately 52 ft long and was installed through sandy soil to socket into a weak limestone formation in Tampa, Florida. Strain gauges were placed at three locations as shown. The top of shaft static load versus deflection response from a rapid load test is shown at top right. The load versus displacement is shown at lower left for the 12-ft long rock socket segment and at lower right for the shaft base (plus 2 ft of side resistance below the lowermost gauges).

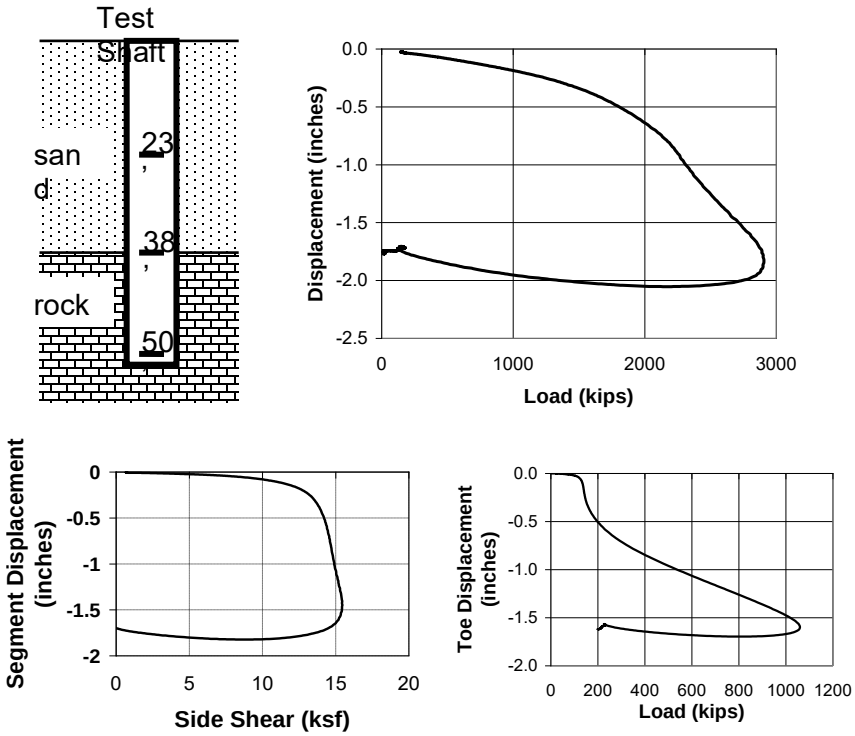


Figure 13-56 Data from an Instrumented Field Load Test

The data derived from the instrumentation provide useful information about the behavior of this shaft that would be impossible to determine from the top of shaft measurements alone. Notice that most of the side shear in the rock is mobilized at less than 1/2 inch of movement and that the side shear exhibits a ductile and strain-hardening response up to the maximum displacement of over 1.5 inches. It is clear from these data that the side shear did not diminish in a brittle fashion at large displacement. The end bearing appears to indicate a soft toe condition in the initial 1/2 inch of displacement, possibly due to incomplete base cleaning, but then increases in a nearly linear manner up to the maximum displacement (1.5 inches is about 3% of the shaft diameter). Additional end bearing resistance could likely be mobilized at even larger displacements; therefore, the test did not achieve an ultimate yield in the geotechnical resistance of this shaft.

A note of caution is in order relative to the precision of unit load transfer interpreted from strain gauge data, as there are several important sources of potential error or imprecision. Even though the gauges may be very precise, the magnitude of the load determined from strain gauges is inherently imprecise because of the effects of shaft diameter and composite modulus of the test shaft. The load transfer is taken as the difference between two imperfect measurements and is therefore subject to even greater imprecision. In all cases, it is necessary to recognize that a reinforced and cast-in-place concrete drilled shaft does not make a precise load cell. Some general guidelines for interpretation of strain measurements in load tested drilled shafts are as follows:

- Four gauges, separated at 90° across the cross section of the test shaft, provides the most reliable indication of average strain used to estimate the load in the shaft. If all four gauges do not obtain similar data, then some judgment regarding the reliability of the data is required. Large eccentricity in the strain profile could indicate a nonuniform resistance on the base or inconsistent stiffness or even gauge malfunctions. Two gauges separated at 180° across the cross section of the test shaft can be used, but eccentricity in the direction orthogonal to a line between these two gauges would not be detected.
- In general, a separation of at least two shaft diameters between measurements is suggested. If strain gauge levels are closely spaced, the relative imprecision in the computed load transfer is likely to be high.
- Strain gauges should generally be positioned to avoid areas of the drilled shaft where uncertainty is likely to be great relative to the test shaft diameter or stiffness, and these parameters affect the load interpreted from a strain measurement. For instance, the zone immediately below a construction casing is often subject to a bulge. At the location of the top of rock there is often a constructed change in diameter which produces a discontinuity and uncertainty in the impedance profile. Near the end of a permanent casing there can be uncertainty related to composite action of the concrete and steel casing which would significantly affect the axial stiffness. All these conditions can influence the strain in the shaft at a given force.
- A separation of at least two shaft diameters is recommended between strain gauges and the loading jacks, particularly for embedded bi-directional load cells. The strain measurements are made near the perimeter of the cross section and are considered to represent the average strain across the shaft; however, at locations near the point of loading it is likely that the strain conditions are not uniform across the cross section.
- The relative distribution of axial side resistance across thin layers of soil or rock with large variations in strength are impossible to distinguish, although the average overall behavior may be measured with a high degree of confidence. Thin is defined as having a thickness that is on the order of the drilled shaft diameter. Rational judgment based upon an understanding of the geology and engineering behavior of the materials is required for interpretation in these conditions. Examples include caliche layers embedded in clay soils, alternating layers of hard and weathered limestone or sandstone, alternating layers of dense sand and soft clays, etc.

- Insufficient displacement to fully mobilize resistance over portions of the length of the load tested shaft may require judgment in the interpretation of the test and the nominal strength used for design.
- The axial resistance can be influenced by residual stresses in the load test shaft from pre-existing forces such as base grouting, concrete shrinkage, installation of nearby piling, or other factors.

In all cases, the engineer must apply judgment to interpretation of the distribution of resistance.

13.2.3.5 Other Instrumentation

Accelerometers are often mounted on or within the shaft for rapid or dynamic load tests. Several different types of accelerometers may be used depending upon the frequency response required for the measurement (high frequency for impact tests, lower frequency for rapid load tests). Acceleration measurement is important to determine inertial resistance of a massive object. The accelerometer measurement may be integrated with respect to time to determine velocity, and integrated again to determine displacement as a function of time. Particle velocity proportional to force is an integral part of the signal matching process used to evaluate high strain dynamic load tests. The displacements obtained from an accelerometer measurement are useful for only short duration changes in displacement and must be referenced to a known displacement; accelerometers cannot directly measure static displacement.

Earth pressure cells are another type of instrument which have been used on rare occasions (usually research) with drilled shafts, usually mounted at the base of a shaft to measure shaft/soil base pressure at various points. Pressure sensors have limited usefulness with load testing because the distribution of pressure over the base of a shaft is not known and may be subject to variations related to soil or rock properties and construction techniques.

13.2.4 Interpretation of Axial Test Results for Design

After completion of axial load tests and determination of values of resistance as described in previous sections of this chapter, the test results must be evaluated for the purpose of estimating geotechnical design parameters. This evaluation must include a consideration of the possible variations in material properties and production shaft behavior across the site, and the effects of any differences between the conditions for design load cases and the tested conditions.

For unit side friction and base resistance, the values determined from the test shaft should be compared with the geomaterial properties at the specific load test location using an appropriate design method. The correlation of axial resistance to geomaterial properties may utilize one of the methods outlined in Chapter 10 of this manual, or as developed from local practice. After this comparison is made, the designer may consider adjustment of empirical design parameters based on the site-specific test data. This “calibrated” correlation may then be used to compute performance of production shafts at other locations with similar geotechnical conditions. This process requires judgment based upon an understanding of the site geology and variability.

Another important issue may arise concerning the interpretation of measured values of resistance when there may be changes in stratigraphy and in-situ effective stress related to scour, grade changes, or fluctuation in groundwater levels. Some of the procedures used in design include empirical correlations which either do not directly account for changes in effective stress or do not account for post-construction reductions in effective stress, so engineering judgment is necessary.

Even in a cohesionless soil, a reduction in vertical stress due to scour would not be expected to result in a proportional reduction in horizontal stress and subsequent axial resistance. The ratio of the change in horizontal stress, $\Delta\sigma_h$, at the shaft/soil interface to change in vertical stress, $\Delta\sigma_v$, due to unloading would be:

$$\Delta\sigma_h = K(\Delta\sigma_v) \qquad 13\text{-}9$$

where K is the earth pressure coefficient for a condition of unloading. Because the soil is unloading, the soil is left with an increased overconsolidation ratio and a final value of horizontal stress, σ_h, which is greater in proportion to σ_v than would be anticipated for the soil prior to scour.

Intact rock formations are not likely to be significantly affected by short term changes in effective confining stress due to the large portion of strength derived from cementation or bond; however, the strength of decomposed rock or frictional joint surfaces within the rock mass may be sensitive to reduction in long term effective stress. Potential effects of future reductions in effective confinement must be evaluated based on an understanding of the geology and rock mass behavior.

13.3 LOAD TESTS TO MEASURE LATERAL RESISTANCE

Drilled shafts are often selected for a project because of their great flexural strength and lateral load resistance. Where the design of the foundation is dominated by considerations of lateral loading, it may be appropriate to consider lateral load tests to validate or improve the design models. This section provides an overview of the most important considerations in planning field load tests to determine lateral resistance of drilled shafts, along with a description of various methods for performing lateral load tests, instrumentation for measurement of the performance of the shaft, and methods for interpreting the results of lateral load tests.

13.3.1 General Considerations in Planning Lateral Load Tests

As with axial testing, the overall objectives must be established and the details of the test program defined with appropriate consideration of the production shafts that the test is intended to model. A discussion of objectives and many of these important details follows.

13.3.1.3 Overall Objectives

The first step in planning lateral load tests is to define the most important objectives of the testing program. For lateral testing, the most important objectives typically relate to measurement or verification of the design soil models that are most critical to the foundation performance. These parameters should be identified during the design process (Block 10 of the overall design process from Chapter 8) by performing parametric studies of the sensitivity of the design to various resistance components. In most cases, the majority of the lateral resistance is derived from the soil resistance of the shallow strata. If the governing design case includes scour, then the test conditions must be established to evaluate the deeper strata. Measurements of the response of the shaft along the length below the point of loading may be very important in order to determine the distribution of lateral soil resistance so that the design can be refined.

It is important that a model be established to evaluate the response of the load test shaft so that the test can be designed to evaluate the lateral resistance of the soil relevant to the production condition. In general,

the conditions of load and moment and rotational restraint at the top of the test shaft are not important. These shaft top conditions will likely differ from the production conditions, but the objective of the test should be to evaluate lateral soil resistance and thus the soil model and parameters used for design. The test shaft may be designed with a greater structural strength compared to production shafts in order to allow a greater mobilization of the lateral soil resistance during the test.

13.3.1.4 Location and Number of Test Shafts

The considerations for location and number of test shafts are similar to those for axial load tests. For a large project, several test shafts may be utilized to perform tests that are representative of different "sites" across the length of the project. Note also that an axial test shaft can be utilized for lateral testing after completion of the axial load test. Lateral tests on an existing shaft are generally economical to perform.

13.3.1.5 GeoMaterial Properties at Test Location

It is very important to determine the properties of the geomaterial that most influence the lateral response of the test shaft at the test location. A pre-test model of the expected behavior of the test shaft should be used to identify the strata that have greatest influence on the response so that the strength and stiffness characteristics of these materials can be investigated and subsequently correlated with the measured lateral resistance. The most important layers for lateral response may be different from those affecting axial resistance.

13.3.1.6 Scour or Changes in Overburden Stress Conditions

Scour and other changes in stratigraphy or in-situ stress conditions can have an even more profound impact on lateral resistance than on axial. If deep scour conditions are anticipated for the structure foundations, a lateral load test in a profile where scour has not occurred may serve only to test the lateral resistance of scourable strata, and will be of little use in calibrating the design model. In such a circumstance, it may be difficult to find an appropriate location to conduct a meaningful lateral load test. Lateral tests may be conducted within an excavated cofferdam as illustrated on the right in Figure 13-57 but the lateral dimensions of the cofferdam may need to be fairly large to provide a representative condition. Lateral testing of a shaft constructed with an isolation casing, as illustrated on the left, is not representative of a condition of scour around the shaft; the overburden soil provides confinement and prevents the development of a passive failure wedge toward the exposed surface. In addition, it may be difficult to isolate the lateral resistance of the soil acting on the isolation casing and through some portion of soil between the casing and the test shaft.

Based on these considerations, it may sometimes not be practical, economical or useful to perform lateral load tests for foundations where deep scour is anticipated, unless a location can be found where similar foundation soils are located reasonably close to the surface. Where it is impractical to conduct lateral load tests, it may be appropriate to assume conservative parameters in the foundation design.

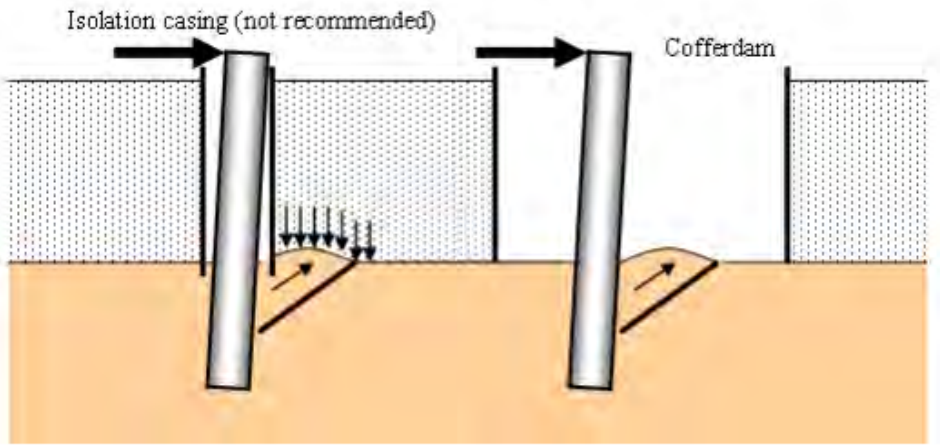


Figure 13-57 Schematic Diagram of Lateral Test where Scourable Overburden is Present

13.3.1.7 Construction of Test Shaft

Construction of a lateral test shaft in a manner similar to that of production shafts is important in several key respects that may differ from axial test shafts. The use of oversized temporary casing and the resulting contact between the shaft and the soil within shallow strata may have little influence on axial resistance but is very important for lateral response. If temporary casing is installed in an oversized hole and then left in place, there could be a gap around the top of the shaft; this gap should be backfilled with grout in a manner consistent with the production shafts. If a temporary casing is left in place, this casing may contribute to the flexural stiffness of the shaft and must be taken into account in the computer model of the load test configuration.

13.3.1.8 Use of Prototype Shafts

In general, prototype test shafts that are smaller in diameter than the production shafts are not recommended for lateral load testing. Smaller diameter test shafts are more strongly influenced by soils at a shallower depth than would be the case for a larger diameter production shaft. In addition, since the flexural stiffness of the shaft is proportional to the 4^{th} power of the shaft diameter, a smaller diameter test shaft does not have sufficient stiffness to transfer stress to the depths that would be influenced by a larger production shaft.

13.3.1.9 Group Considerations

Considerations for groups of drilled shafts subject to lateral load are described in Chapter 11, and typically include some modification of the lateral p-y response to account for group effects from overlapping zones of passive soil resistance. Since a lateral load test is most often performed on an isolated shaft, the application of the results to design of production shafts in a group would require modification of the lateral resistance model derived from the test. The shafts in a group will transfer a greater portion of the load to deeper strata compared to the isolated test shaft because of the reduced soil resistance attributed to group effects.

13.3.2 Test Methods and Procedures

Most lateral load tests are conducted as conventional static tests by jacking the test shaft laterally with a hydraulic cylinder. However, there are circumstances where other methods of lateral testing have advantages. The following sections describe the basic methods used to perform conventional lateral load tests on drilled shafts as well as rapid and bi-directional testing methods.

13.3.2.3 Conventional Static Lateral Load Test

A conventional lateral load test is commonly conducted by either pushing the test shaft away from one or more reaction shafts or piles or pulling it toward the reaction. The typical test set-up and procedures for this type of test are described in ASTM test designation D 3966-07. The load is applied either by a jack that pushes on the test shaft at ground level away from a reaction system, or by a jack connected to the side of the reaction system opposite to the test shaft (or *vice versa*) and attached to a cable or tie that allows the test shaft to be pulled toward the reaction. The load is measured by a load cell that is positioned adjacent to the jack in a manner similar to that for axial load tests. If two shafts are jacked apart (or together), it is possible to obtain measurements on both shafts simultaneously.

A photograph of a typical arrangement for a lateral loading test of a drilled shaft is shown in Figure 13-58. In this case, the test shaft is a vertical, 30-inch diameter shaft that is being jacked away horizontally from a steel reaction beam that spans between two other reaction shafts of the same size. In this particular test, in stiff clay, the drilled shaft was pushed approximately 3 inches by a ground-level shear load of about 50 tons.

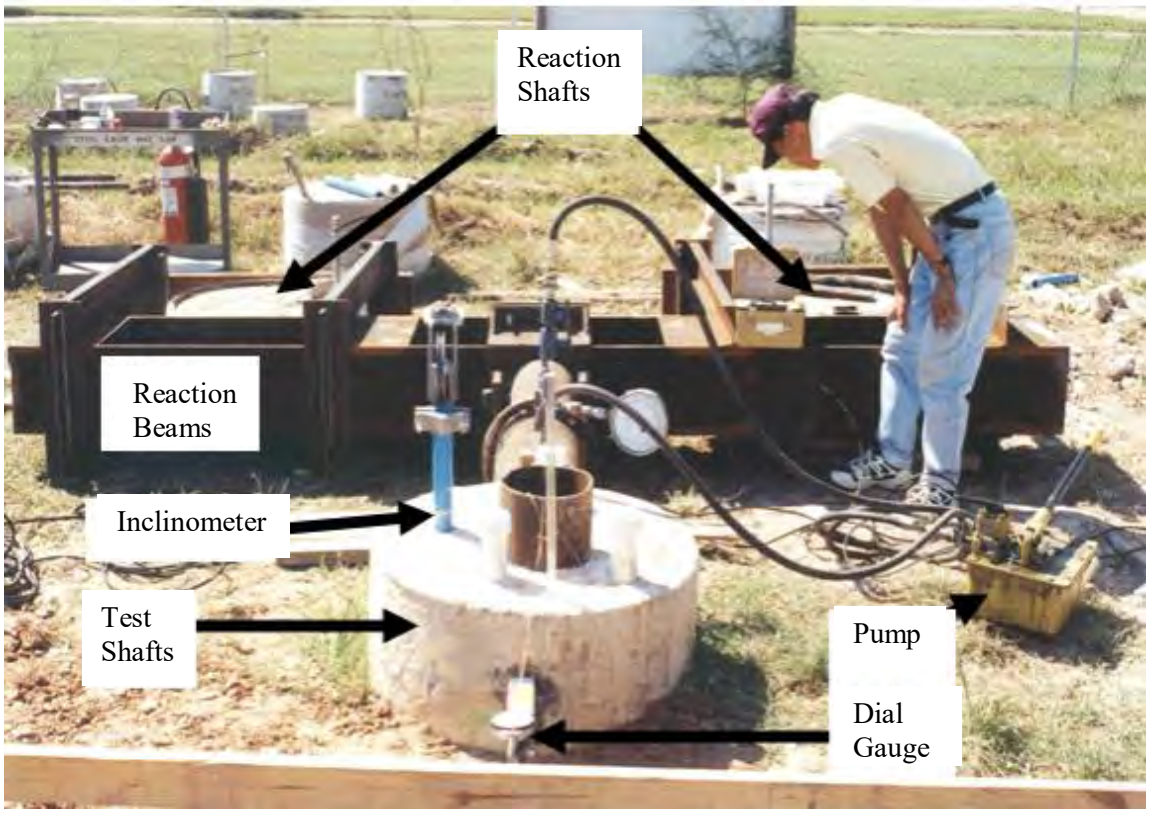


Figure 13-58 Conventional Static Lateral Load Test Setup

Several aspects of the lateral test setup are very important for both reliable test information and for safety during the test. Because the test and reaction shafts are subject to rotation at the top of the shaft, a change in alignment in the hydraulic loading system during the test is inevitable. The system must include a suitable bearing or clevis bracket to accommodate this rotation without imposing eccentricity in the jack or load cell, and without risk of a bearing plate popping out of the system. If two shafts are pulled toward each other, a sufficient distance is needed between the shafts to assure that the passive soil zones do not overlap and affect the results.

The photo in Figure 13-59 illustrates a clevis bracket mounted to the end of the hydraulic ram, with the load cell mounted between two beams and the beam system attached to the test shaft. This test shaft was constructed by Kansas DOT with a rectangular extension above grade and a flat steel plate mounted into the extension for ease of connecting to the loading system.

The lateral test setup shown in Figure 13-60 includes provisions for performing short term, two-way cyclic lateral loading, which might be an important consideration if the governing load case includes seismic or other extreme events that can produce cyclic loading (Brown et al, 2002). This system includes a large hydraulic pump with a hydraulic loading ram that is programmed to follow a controlled deflection versus time curve. The loading system also included a pinned connection to the two shafts, one of which was encased with a permanent steel liner at the surface. A steel casing provides a convenient surface for welding connections to the test shaft and increases the flexural stiffness of the shaft.

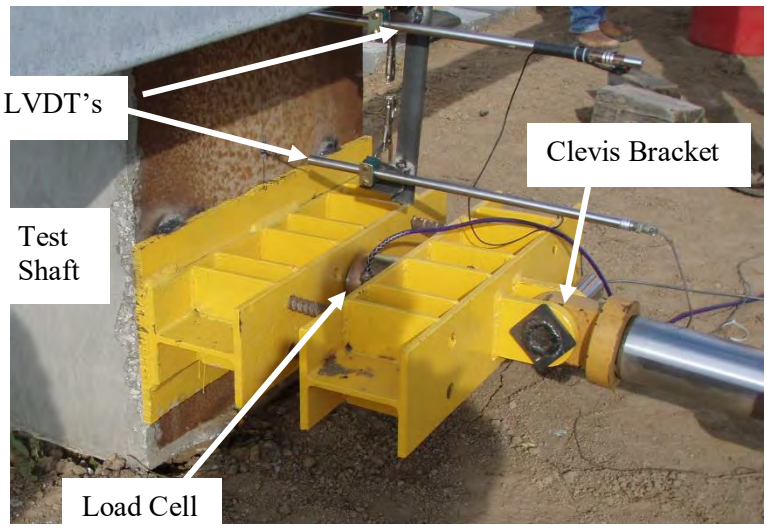


Figure 13-59 Clevis Bracket and Load Cell Mount for Lateral Loading

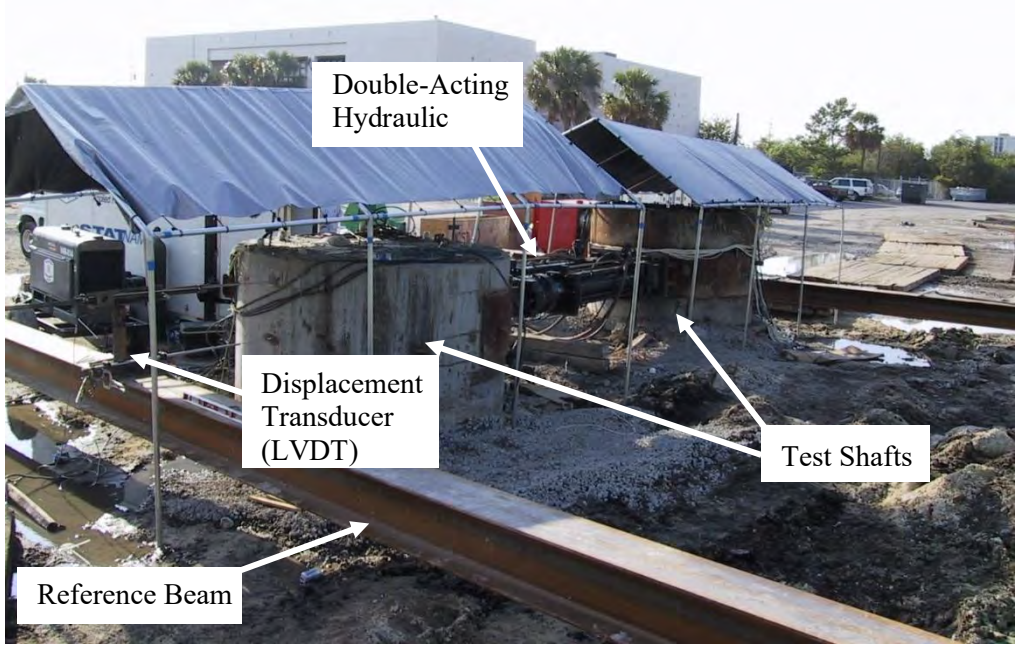


Figure 13-60 Cyclic Lateral Load Test Setup

13.3.2.4 Bi-Directional Testing

Bi-directional lateral load tests have been performed by inserting a load cell vertically into a rock socket, casting concrete around the cell and using the cell to jack the two halves of the socket apart (O'Neill et al., 1997). The lateral load applied to the geomaterial per unit of socket length is easily computed by dividing the load in the cell by the length of the test socket, which is approximately the length of heavy steel plates that are attached to each side of the cell. Two arrangements for such a test are shown in Figure 13-61. The cell at left was installed into a 6-ft diameter socket into sandstone at a depth of around 100 ft below the surface. The pair of cells in the photo at right were installed into a socket which was 8-ft diameter and 15-ft long into a chalk formation approximately 60 ft below grade. The steel beams connect the two cells which were jacked independently to maintain equal lateral displacement along the length of the socket and to split the shaft vertically, pushing the two halves apart. Lateral displacement is measured by using sacrificial LVDTs that connect between the plates and measure the opening between the plates.

This test is not a direct simulation of a laterally loaded drilled shaft in flexure, but rather a type of in-situ test of the lateral resistance of the geomaterial within a specific formation subjected to a load of a size similar to a drilled shaft. The method allows the site-specific evaluation of the lateral response of a deep stratum which might be subsequently subjected to lateral loads from drilled shafts, including shafts in a large group or after exposure of the formation from scour. The interpretation of test results must include an evaluation of the effects of removal of overburden or scour effects, as discussed in other sections of this chapter.



Figure 13-61 Bi-directional Lateral Testing Apparatus

13.3.2.5 Rapid Load Test

Drilled shafts have also been tested by mounting a rapid load testing device on skids horizontally adjacent to the shaft and loading with a pulse of ground-level shear (O'Neill et al., 1997; Rollins et al., 1997). This type of test can be more economical than the conventional test because the need for a reaction system is avoided. The system, illustrated in Figure 13-62 is capable of applying lateral loads in excess of 1,000 tons, thus providing a capability of testing full size shafts of large diameter. Rapid loading may also be more appropriate than the conventional test when the type of design loading being considered is a rapid or impact loading (such as seismic, vessel or ice impact). A method of analysis of lateral rapid load tests is described by Brown (2007) which includes a means of accounting for inertial and rate of loading effects.

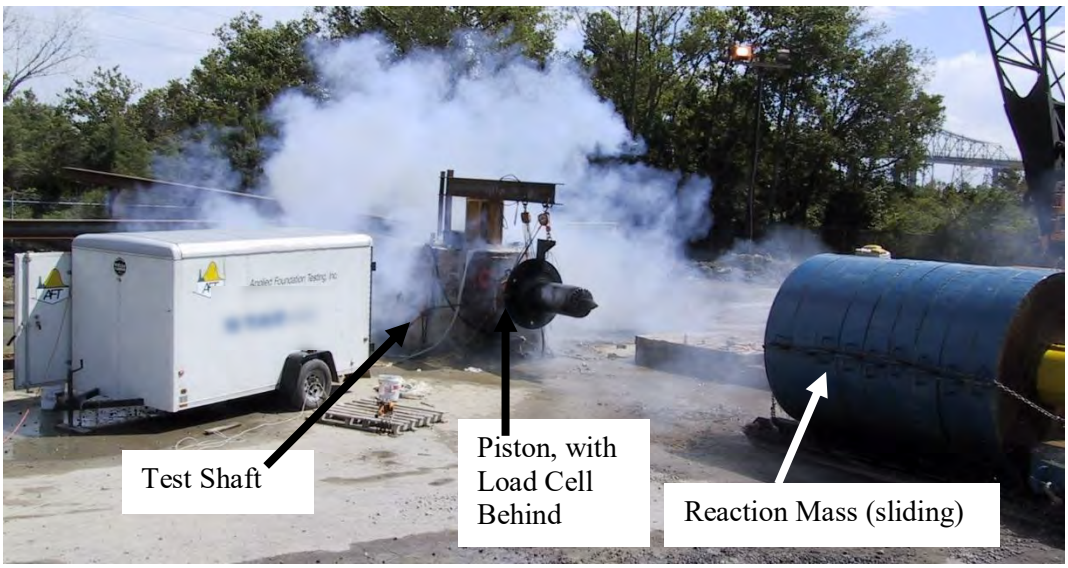


Figure 13-62 Rapid Lateral Load Test

13.3.3 Instrumentation

Much of the instrumentation used for lateral load testing is similar or identical to that used for axial load tests, as described in previous sections of this chapter. Some aspects of instrumentation specific to lateral load testing are described below.

13.3.3.3 Measurement of Load

Load is applied and measured with hydraulic jacks and load cells in conventional lateral load tests similar to that for axial tests with a couple of noteworthy differences. The jack must operate in the horizontal direction without leaks or problems related to the orientation of the jack, and the connection to the test shaft must be capable of accommodating a significant amount of rotation as described in Section 13.3.2.1. The magnitude of the load is often smaller, but the magnitude of the displacement in a typical lateral test may be much larger than in a vertical test.

Load cells are preferred for measurement of the load, with backup from the hydraulic pressure on the jack. For the rapid lateral loading using the combustion gas pressure device, a load cell is incorporated into the system. For bi-directional testing using the bi-directional load testing device, the pressure in the cell is calibrated to load.

13.3.3.4 Measurement of Shaft Displacement

Lateral displacement of conventional lateral static and rapid load tests may be measured using displacement transducers (dial gauges or LVDTs) mounted on a reference beam. Where large lateral displacements are anticipated, these may need to be of unusually large travel, such as the LVDT shown in Figure 13-63. Long travel linear potentiometers (a variable resistor) may also be used for this purpose; some of these units are fabricated with a spring-loaded tension wire which can be extended to measure movements of several feet. Horizontally oriented accelerometers may also be mounted on the test shaft to measure lateral displacement during a rapid lateral load test.

It is important to obtain measurements of lateral displacement along the length of the shaft to properly interpret the lateral test results, and such measurements are easy and inexpensive to obtain. A grooved inclinometer casing can be incorporated into the longitudinal reinforcing cage to provide access. A pre-test measurement of the vertical profile of the shaft is obtained using an inclinometer sensor (instrument for measuring slope for a near-vertical orientation) and used as a baseline for the test shaft in the same way that this instrument might be used in slope stability studies or for measurement of lateral movement of a wall. The lateral displacement relative to the bottom of the shaft can then be obtained during the test using the inclinometer, as illustrated in Figure 13-64(a). Note that for a relatively short test shaft, the base of the inclinometer casing (the reference point for lateral displacement determination located near the base of the shaft) may not remain at zero displacement. In such a case, the relative displacements must be adjusted to a known point above grade at the location of the reference system measurement. In order to obtain the maximum sensitivity of the instrument, the casing should be positioned so that one set of internal grooves align with the direction of lateral loading. Also, it is desirable for the casing to be positioned at the neutral axis of the shaft to avoid or minimize axial strain effects on the inclinometer readings.

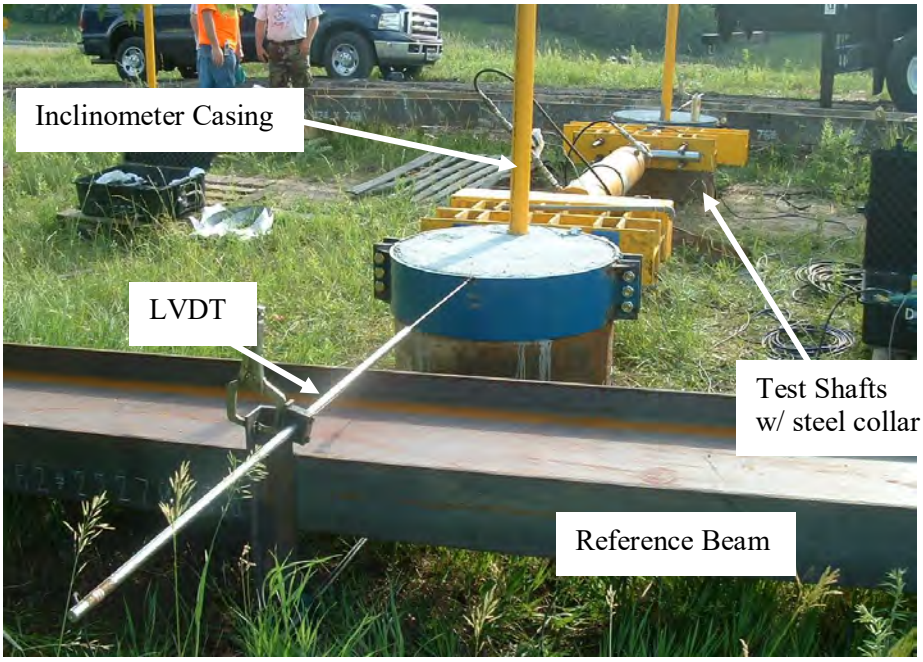


Figure 13-63 Long Travel Displacement Transducer and Inclinometer Casing



(a) Inclinometer Profiling (b) Shape Accelerometer Array

Figure 13-64 Measurements of Lateral Displacement in a Lateral Test Shaft below Grade

To obtain lateral displacement measurements with an inclinometer, it is necessary that the loading be stopped and held at a constant displacement during the 20 to 30 minutes that may be required to perform the inclinometer profiling.

A newer and faster method to obtain these measurements is to suspend a string of shape accelerometer arrays (Rollins, et al., 2009) in the casing, as shown in Figure 13-64(b). These segments each contain triaxial, micro electro mechanical system (MEMS) accelerometers which provide a measurement of the

bend angle between each segment. With these measurements, the displaced shape of the drilled shaft can be continuously monitored during the lateral load test.

Accelerometers may be used within an inclinometer casing during a rapid lateral load test to obtain a measurement of lateral motion at various points below the top of the shaft. An illustration of this instrument is provided in Figure 13-65. The wheels align with the grooves in the inclinometer casing and the mount allows the accelerometer to be oriented in the direction of loading. The string of down-hole accelerometers are monitored with the data acquisition system used to monitor the other instrumentation during the rapid lateral load test.

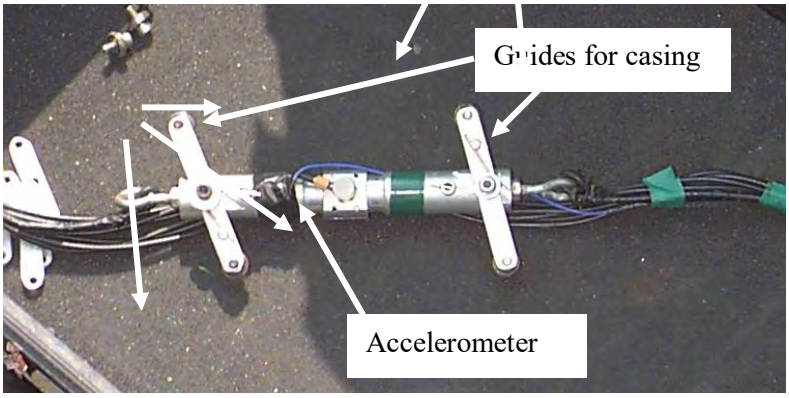


Figure 13-65 Down-hole Accelerometer for Displacement Measurement During Rapid Lateral Load Test

13.3.3.5 Measurement and Interpretation of Strain

Strain measurements during a lateral load test may be obtained using the same type of sister-bar mounted gauges described in Section 13.2.3.3. However, the objective of strain measurements during a lateral test is to obtain a measurement of the bending moments within the shaft. If gauges are mounted on both the far and near sides of the shaft in the direction of lateral loading, measurements of compression and tension are obtained as illustrated in Figure 13-66.

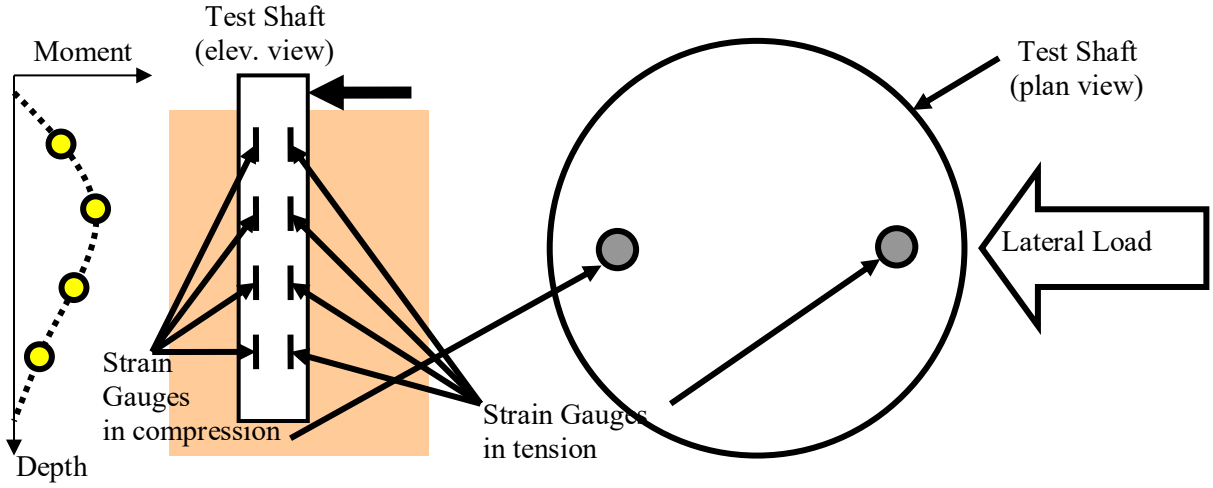


Figure 13-66 Strain Gauges to Measure Bending Moments

For a simple elastic beam, the bending moment, M, is obtained from measured strain, ε, by:

$$M = \frac{\varepsilon EI}{c} \qquad 13\text{-}10$$

where: E = elastic modulus
I = moment of inertia
c = distance from neutral axis to the point of strain measurement

The tension and compression strains measured at equal distance from the center of an elastic beam in pure bending would be equal in magnitude and opposite in sign. A reinforced concrete drilled shaft does not behave as a simple elastic beam when subject to bending because the concrete will crack in tension and, as a result, the neutral axis shifts toward the compression side. The composite EI of the shaft changes dramatically with cracking, as described in Chapters 9 and 12. These chapters outline a method of estimating the nonlinear EI of the cross section for design purposes, and the methods described are generally conservative with respect to estimating the flexural stiffness (i.e., the EI is unlikely to be less than computed). In a real measurement of a test shaft, the actual EI is sensitive to:

- The "as-built" concrete compressive strength, which may be higher for the concrete cured in-situ than that of the cylinders made for quality control purposes,
- The actual tensile stress at which the concrete cracks, which may be higher than the relationships with compressive strength assumed for design,
- The actual concrete modulus, which may be higher than the typical relationship with compressive strength assumed for design,
- The actual "as-built" diameter of the shaft at the location of measurement,
- The effect of confinement provided by the soil or rock on the cracking in the shaft,
- The actual location of the reinforcing cage relative to the center of the shaft (if shifted toward the tension side, the reinforcing has a greater effect),
- The actual tensile strength of the reinforcement, which is normally higher than the minimum value specified for the grade of reinforcing steel used,
- The flexural strength, stiffness, and composite action of a steel casing left in place, even if the casing were not considered a part of the structural design of the shaft.

In summary, the nonlinear flexural strength and stiffness of a reinforced concrete column which is cast-in-place in a drilled hole makes unreliable the correlation between bending strain measurements and actual bending moments in the shaft. Bending strain measurements are useful in a lateral load test of a drilled shaft because they can help to identify the onset of cracking in the concrete (the tensile strains will suddenly increase relative to the compressive strains) and the location of the maximum bending moment and flexural yielding in the shaft.

13.3.4 Interpretation of Lateral Test Data

The interpretation of the results of a lateral load test requires the use of a computational model as described in Chapter 9. A model of the lateral load test with the appropriate known boundary conditions at the top of the shaft (typically a zero moment and a known shear force) must be established with a best estimate of the actual shaft diameter and nonlinear flexural stiffness (EI). In the experience of the writers, the actual flexural stiffness is often somewhat higher than that computed using the methods outlined in Chapter 9 for

the reasons cited in the previous section. The model is used to compute (a) the load versus deflection response at the top of the shaft for each of several loads for which measurements have been obtained, and (b) the deflection and bending moments versus depth below top of shaft for each of these loads.

The actual measurements of load versus deflection and deflection versus depth for each load are then compared to the model predictions, and the soil strength and stiffness characteristics (implemented through the p-y soil or rock model used for design) are adjusted as required to obtain a reasonable agreement. An overestimate of displacements in the model would reflect a conservative soil model. The shaft top movements in the model might fortuitously indicate good agreement with the measured deflections at the top of the shaft, but if the displacement profile below grade is not in agreement then the distribution of soil resistance in the model is not correct.

It should be noted that an underestimate of the flexural stiffness of the as-built test shaft could lead to an overestimate of the strength and stiffness of the soil model. If the shaft response is stiffer than expected because the flexural stiffness of the test shaft is better than expected, it might be easy to erroneously attribute this improved response to stronger soil. Therefore, it is imperative that the flexural strength and stiffness of the test shaft be appropriately modeled.

Once the designer is satisfied that the soil model provides reasonable or conservative agreement with the test measurements, the model can be used with the appropriate boundary conditions for production shafts as outlined in Chapter 9. Variations in stratigraphy across the site should be determined by the site exploration and included in the model for each foundation as part of the final design process.

If lateral load tests are considered for a project, they should be performed during the project design phase. Considering the time constraints during construction, there likely would be insufficient time available to modify the foundation design to incorporate the findings of a construction-phase lateral load tests program. If a construction-phase lateral load test program is required, the foundation design should be reasonably conservative with respect to lateral soil properties in order to minimize the risk of construction delays resulting from unexpected load test results.

13.4 SUMMARY

This chapter provided an overview of load testing techniques, measurements, and strategies for interpretation of results for design purposes. Because of the great advances in load testing technologies in recent years, field load tests offer designers a great opportunity for improved economy and reliability of drilled shaft foundations if the tests are used effectively and interpreted appropriately.

CHAPTER 14
DRILLED SHAFT CONSTRUCTION SPECIFICATION

14.1 INTRODUCTION

Appendix D provides a guide specification for the construction of drilled shaft foundations. This specification is adapted primarily from "Section 5 - Drilled Shafts" in the AASHTO LRFD Bridge Construction Specifications, 4th Edition (AASHTO, 2017b), and also includes provisions reflecting the authors' recent experience with the installation and testing of drilled shaft foundations. The guide specification includes commentary that provides further guidance and possible alternative provisions to assist engineers in modifying the guide specification to suit specific project conditions. The base specification and commentary from AASHTO are presented in standard font. Supplementary provisions and commentary from the authors and other sources are shown in italics.

The guide specification is not intended to be used directly for project applications. Similar to other guide specifications, the specification presented in Appendix D may not be completely suitable for the specific conditions encountered at a particular project. Considering the wide variability in project requirements for drilled shaft foundations, engineers should develop their own project-focused specification using the specification in Appendix D only as a guide for identifying issues and provisions typically applicable to drilled shaft construction.

In addition to this guide specification, the design engineer should also consult other reference specifications for further assistance in developing specification provisions applicable to the unique conditions relevant to any particular project. These reference specifications may include:

- Specification for the Construction of Drilled Piers, ACI 336.1-01 (ACI, 2017)
- Standards and Specifications for the Foundation Drilling Industry (ADSC, 1999)
- Standard Specifications from government agencies, including state departments of transportation.

This chapter provides a general discussion of several key topics that are addressed in the FHWA guide specification and that should be considered during the preparation of any project-specific drilled shaft construction specifications.

14.2 DESIGN CONSIDERATIONS

A primary consideration in the development and application of drilled shaft specifications for any project is the relationship of drilled shaft performance to the means and methods used for construction. Often, the resulting nominal resistance of a drilled shaft and the serviceability characteristics of the completed shaft will be influenced by the method of drilled shaft installation as well as the care exercised in performing the work. Some examples illustrating this close relationship between construction techniques and drilled shaft performance include: poor bottom cleaning procedures that may considerably reduce the available base resistance; prolonged exposure of the sides of the drilled shaft excavation to slurry that may greatly reduce the available friction resistance; and failure to maintain a stable shaft excavation that may result in voids or soil inclusions in the shaft concrete that may compromise the structural integrity of the completed drilled shaft; to note just a few. Unlike driven piles, where the resistance to driving provides a means of assessing the load bearing resistance of each pile, the proper performance of drilled shafts relies heavily on the consistent and repeatable application of the drilled shaft installation

procedures demonstrated by the successful performance of technique shafts (also sometimes called trial, demonstration or method shafts) and test shafts. Accordingly, for drilled shaft specifications, emphasis must be given to:

- Qualifications of the drilled shaft contractor and key personnel assigned to drilled shaft construction,
- Proper planning of the work through the preparation and review of a Drilled Shaft Installation Plan.
- The application of appropriate drilled shaft installation methods for the anticipated ground and groundwater conditions,
- Use of technique shafts to evaluate the contractor's selected means and methods of drilled shaft construction,
- Use load test shafts to verify design assumptions and justify use of higher resistance factors.
- Integrity testing to verify the required structural integrity (see Chapter 16), and
- Clearly defined acceptance criteria and installation tolerances.

These and other specification topics are further discussed below.

14.3 QUALIFICATIONS OF DRILLED SHAFT CONTRACTORS

Drilled shafts are typically highly loaded, and are frequently non-redundant elements used for structural support. To assure reliable performance, such elements require the high degree of workmanship which can be provided only by experienced drilled shaft contractors. Minimum drilled shaft contractor qualifications should be required by the contract documents to assure prior contractor experience. In addition, when appropriate, the specifications should require the successful installation of a technique shaft or shafts at the start of construction to demonstrate the contractor's construction capabilities and proposed installation method relative to the existing subsurface conditions at the project site. The degree of risk and complexity of a particular project should be considered when establishing contractor qualifications for the project. The guide specification in Appendix D contains suggested qualification requirements for a typical transportation project using drilled shaft foundations.

The specification should also require minimum qualifications for the key contractor staff responsible for installation of the drilled shafts. These may include the drilled shaft superintendent, foremen and lead drillers, as well as testing personnel and independent drilled shaft inspectors (for design-build contracts).

It is generally undesirable for a general contractor to hire a drilled shaft specialty contractor only to excavate the drilled shaft, while assigning the placement of steel and/or concrete to a different organization. The necessary additional coordination effort and any associated delay in the placement of concrete may impact the performance of the completed drilled shafts. Also, separating these operations invites disputes between the drilled shaft contractor and the prime contractor whenever there is a question regarding the integrity of the completed shaft (e.g., was a defect caused by improper concrete placement procedures or by use of an inappropriate concrete mix?). Construction of a drilled shaft includes excavation, placing steel and placing concrete; these are all critical operations that should typically be performed in a continuous and coordinated manner by a single, qualified contractor.

14.4 CONSTRUCTION METHOD

The construction methods to be permitted on a specific project (Chapters 3, 4 and 5) must consider the ground and groundwater conditions at the project site, and the design requirements for ground resistance

(i.e., reliance only on side resistance, only end bearing resistance, or combined side resistance and end bearing). These factors will largely influence the type of drilling equipment used, the method of excavation (wet or dry), the type of drilling fluid, the use of permanent or temporary casing, and shaft cleanout criteria, among others. The drilled shaft construction methods, as well as design requirements, will then influence the concrete mix design and the method of placement. For wet excavations, for example, the specification must include provisions requiring a concrete mix with necessary slump or spread range, minimum workability duration, and maximum aggregate size suitable for tremie placement.

Fortunately, numerous combinations of equipment and procedures are available to successfully install drilled shafts in any ground condition and to meet all design requirements. The specification, therefore, should not needlessly restrict contractors in their choice of tools, equipment or construction method. To achieve a cost-effective project, the specification must permit as much flexibility to the contractor as possible, within the constraints of the project and the existing site conditions. However, when site conditions dictate the need for certain installation methods, such as the use of wet methods for installation of drilled shafts in cohesionless soils below the groundwater level, it would be appropriate to include such general requirements.

14.5 DRILLING FLUID

Drilling fluid is an effective means of stabilizing drilled shaft excavations until either a casing has been installed to a competent layer or until the concrete is placed, as discussed in Chapter 5. The type(s) of fluid (mineral slurry, polymer slurry or water) specified should be appropriate for the anticipated ground conditions and selected installation method.

The properties of drilling slurry should be monitored and controlled during slurry mixing, during drilling, and also just prior to concrete placement. Primary concerns in slurry use are: 1) the stability of the borehole should be maintained during excavation and concrete placement, 2) the slurry does not weaken the bond between the concrete and either the side of the drilled shaft or the steel reinforcement, 3) all of the slurry is displaced from the drilled shaft excavation by the rising column of concrete, and 4) any sediment carried by the slurry is not deposited in the drilled shaft excavation or rising concrete.

The performance and effectiveness of the drilling fluid can be achieved through appropriate specification provisions, including: 1) identifying the types of drilling fluids that may or may not be used, 2) specifying a suitable range of slurry properties both during excavation and just prior to concrete placement, 3) performing slurry inspection tests, and 4) construction of pre-production technique shaft(s) to evaluate the contractor's proposed wet excavation method.

14.6 LOAD TESTING

Load testing of drilled shafts, as discussed in Chapter 13, is often specified for projects that require high capacity foundations for major transportation structures, for ground conditions in which geotechnical resistance cannot be reliably estimated, and for projects where the number of drilled shafts may justify the cost of a load test program.

Considering the generally high axial resistance typical of drilled shaft foundations, the use of conventional static load tests using a dead weight or reaction frame to load the top of the drilled shaft is generally not practical. Bi-directional load cell testing, in which a bi-directional load is applied by a hydraulic jacking mechanism cast within the drilled shaft, is a more practical and economical method for

determining the axial resistance of a high capacity drilled shaft, and currently is a commonly used method for load testing drilled shafts. The procedures for conducting this type of load test are presented in ASTM test designation D8169/D8169M for "Standard Test Methods for Deep Foundations under Bi-directional Static Axial Compressive Load." This new ASTM standard is referenced in the guide specification in Appendix D.

Drilled shaft test programs have also included rapid, compression force pulse tests and high strain dynamic tests for estimating the axial resistance of drilled shafts. Each of these methods estimates resistance along the drilled shaft by evaluating the dynamic response of the drilled shaft and the dynamic characteristics of the supporting ground. Rapid, compression force pulse load tests are addressed in ASTM test designation D7383 for "Standard Test Methods for Axial Compression Force Pulse (Rapid) Testing of Deep Foundations." This standard covers load tests where the test load is applied as an axial compressive force pulse at the top of the drilled shaft using either a combustion gas pressure (ASTM "Procedure A") or using a cushioned drop mass ("Procedure B"). Procedures for performing high strain dynamic load tests are provided in ASTM test designation D4945 for "Standard Test for High Strain Dynamic Testing of Deep Foundations." The guide specification presented in Appendix D references these ASTM standards when either of these types of load tests are specified or considered.

Lateral load tests are occasionally performed on drilled shafts to determine the response of the drilled shafts to lateral loading, or to determine the p-y resistance properties of the surrounding geomaterials. Procedures for performing lateral load tests by the static load method should be in accordance with the requirements of ASTM D3966/D3966M for "Standard Test Methods for Deep Foundations under Lateral Load." At the present time there are no standards for performing lateral load tests by the bi-directional loading method or the rapid load test method, as described in Chapter 13. Accordingly, until such standards are developed, caution must be exercised in the planning, execution and interpretation of the results of lateral load tests using these methods.

14.7 INTEGRITY TESTING

Drilled shafts can experience construction defects such as necking, bulging, voids, honeycombing, loss of concrete cover, etc., particularly when drilled shafts are installed using the wet method of construction. Therefore, non-destructive integrity testing, as described in Chapter 16, should generally be a part of any drilled shaft foundation project that anticipates drilled shaft installation using the wet method of construction. It should also be included as an essential element of all technique shafts and load test shafts. However, integrity testing should not be relied upon as the sole means for assessing the structural adequacy of the drilled shaft, but must be considered, along with proper design of the drilled shaft, proper shaft installation, and thorough construction inspection, as a key component of the overall QA/QC program, as discussed in Chapter 17.

Integrity testing may be specified for some or all the following purposes, depending on the details of the drilled shaft foundation design:

- at the start of construction, to confirm the suitability of the contractor's proposed shaft installation methods
- routinely on all production drilled shafts, or on a defined percentage of production drilled shafts, as part of the project QC/QA program
- to evaluate the structural integrity of a drilled shaft whenever there is a significant change to the contractor's means and methods of shaft installation

- to investigate drilled shafts that are suspected of having defects to the shaft concrete based on observations during shaft installation
- to evaluate the effectiveness of remedial measures performed on a drilled shaft

As discussed in Chapter 16, the methods available for integrity testing include both internal integrity test methods (Cross-hole Sonic Logging, Thermal Integrity Profiling, and Gamma-Gamma Density Logging) and external integrity test methods (Sonic Echo and Impulse Response). Chapter 16 also provides guidance for the selection and application of these non-destructive integrity testing methods.

The guide specification in Appendix D includes provisions for CSL testing since this is a commonly used test method and one that requires installation of tubes within the shaft during shaft construction. The guide specification also includes provisions for performing Thermal Integrity Profiling, as this method is currently gaining acceptance and application in practice. If internal testing is not specified, consideration should be given to including in the construction specifications provisions for performing external integrity testing by a qualified testing specialist to evaluate drilled shafts suspected of having defects in the shaft concrete based on observations during concrete placement operations.

Where owner agencies require and specify specific types of integrity testing for drilled shafts (e.g., Gamma-Gamma Logging for Caltrans projects), these owner requirements should be incorporated or included by reference in the drilled shaft specification.

14.8 CONSTRUCTION PHASE SUBSURFACE INVESTIGATIONS

As discussed in Chapter 2, the design phase subsurface investigation program should be tailored to address the specific requirements of the drilled shaft foundation design and the existing site conditions. The data from the design phase investigation program should be referenced or included in the contract documents to obtain responsive bid prices for the work, and to reduce the risk of unanticipated conditions and differing site condition claims during construction (see Section 2.6). However, situations often arise where additional subsurface investigations will be required as part of the construction contract. Such situations may include cases where there are non-redundant drilled shafts that require a confirmation boring at each shaft location; for shafts in rock to determine the specific depth and length of the rock socket at each shaft; and for sites with highly variable subsurface conditions that may not have been sufficiently defined during the design phase investigation program. Construction phase subsurface investigations are also commonly required for design-build contracts since the locations and details of design elements are determined by the design-builder, and typically cannot be defined during the contract procurement process.

The detailed requirements for performing construction-phase subsurface investigations are typically contained in a separate specification section which is then referenced in the section on Drilled Shafts. Guidelines for planning and implementation of subsurface investigations are provided in FHWA Geotechnical Engineering Circular No. 5 (FHWA-NHI-16-072 by Loehr et al., 2017), and discussed in Chapter 2.

14.9 DRILLED SHAFT INSTALLATION PLAN

An essential element of any project that requires the installation of drilled shaft foundations is a Drilled Shaft Installation Plan. The Drilled Shaft Installation Plan effectively communicates to drilled shaft construction personnel and field inspectors the project requirements for drilled shaft installation. The

Plan also serves as a basis for identifying any equipment or procedures that deviate from those previously reviewed and accepted for installation and testing of the drilled shafts.

The Drilled Shaft Installation Plan is prepared by the contractor and submitted for the review of the owner and/or foundation design engineer in advance of initiating any drilled shaft construction. The guide specification in Appendix D provides a list of the minimum information that needs to be included in a Drilled Shaft Installation Plan. The Plan should be specific to the particular project and not just a standard list of general steps typical of drilled shaft construction. The Plan should fully document the equipment and procedures to be used in the work, allowing the engineer opportunity in advance of construction to confirm compliance of the contractor's proposed equipment and procedures with the contract documents; and to identify proposed practices that may adversely influence the nominal resistance or serviceability characteristics of the completed shafts

Following receipt of review comments, the contractor will revise and re-submit the Plan. Work at the drilled shafts should not proceed until the Plan is accepted.

During the course of the work, the Drilled Shaft Installation Plan should be updated and resubmitted to the owner and/or the foundation design engineer for review and approval whenever there is a significant modification to the drilled shaft installation equipment or procedures. Field staff should always be provided with the latest approved version of the Plan to assure that the work is performed as intended.

14.10 MEASUREMENT AND PAYMENT

Unit pricing is the typical method for measurement and payment for drilled shafts constructed under a design-bid-build contract. With unit pricing, two alternate methods of payment for drilled shaft excavation can be specified, including unclassified excavation and separate payment for soil and rock excavation. The preferred approach, and the one reflected in the guide specification in Appendix D, includes separate pay items for soil and rock excavation.

Unclassified Excavation: Unclassified excavation may be appropriate when little or no hard rock is anticipated on a project. It may also be a more practical approach in cases where the transition from geomaterials that can be drilled with standard techniques at a rapid rate to geomaterials that are difficult to excavate is hard to define or ambiguous. Unclassified excavation may also be preferred by some contracting agencies to reduce the administrative effort in tracking the separate pay items.

Specifying unclassified excavation may result in an increased contingency cost in contractor bids since the contractor assumes the risk of an increased length of difficult drilling. This concern can be addressed, but not completely resolved, by performing an appropriate design-stage site investigation and making this information available to the bidders.

Separate Payment for Soil and Rock: Separate measurement and payment for soil and rock excavation provides an equitable means of payment for excavation of hard rock that may require different excavation equipment and/or result in considerably slower excavation rate. This approach reduces much of the uncertainty in the contractor bids, and is therefore generally more economical than unclassified payment for excavation.

Separate pay items for soil and rock excavation should be defined in terms of the difficulty and rate of excavation. Often this is a subjective decision, related to the equipment used for excavation. To address this issue, the guide specification suggests using an Unconfined Compressive Strength of 1,000 psi to define the division between soil excavation and rock excavation. Using this approach, a sufficient

number of strength tests must be performed prior to and during construction to determine the strength of the various rock strata to be encountered.

An alternative approach used by some contracting agencies is to differentiate excavation pay items by the equipment needed to advance the hole. With this approach, standard or soil excavation includes hole advancement with conventional augers and drilling buckets. Special or rock excavation is paid when the hole cannot be advanced with conventional tools, but requires special rock augers, core barrels, or other methods of excavation using unconventional tools.

When separate excavation pay items are used, the specification needs to carefully define the materials and/or excavation conditions (tools, rate of advance, etc.) that will be used to determine the appropriate pay item for shaft excavation. This approach also requires inspectors knowledgeable in classifying geomaterials and in shaft excavation methods to determine the appropriate pay item to apply as the excavation advances. In addition, this approach requires increased coordination with the contractor to obtain concurrence on pay quantities and to address disagreements that may arise regarding material classification.

Obstructions: Whether shaft excavation is paid as unclassified excavation or as separate items for soil and rock excavation, the specifications should include an additional pay item for obstruction removal. Obstructions are usually paid under a separate item since they typically require unconventional excavation techniques and interrupt the general excavation operations. Some agencies pay for obstruction removal at a factor of 2 or 3 times the rate that was bid per unit of depth based on soil drilling, some pay based on a report of the contractor's time and materials, and some use other methods. One approach may be to include a defined quantity for obstructions, measured by length of obstructions in the shaft excavation, and payment made separately for obstruction removal only when the length of obstruction drilling exceeds this specified value; this approach shares the risk of obstructions between the owner and the contractor, but still requires tracking of all work related to obstructions.

The Washington State DOT (2018) standard specification for drilled shafts includes provisions for payment, at an agreed hourly stand-by rate, for construction equipment that is idled during the period needed for obstruction removal, provided that the equipment cannot be reasonably reassigned within the project.

Steel Reinforcement and Concrete: Steel reinforcement and concrete can be measured separately based on the weight of steel and the volume of concrete used in the shaft. Alternatively, these items can be combined and paid for based on a price per unit length of shaft, for each diameter shaft used on the project.

If paid on a unit volume basis, the quantity of concrete is typically determined from the design dimensions of the drilled shaft, and excludes any excess amount used for oversized excavations, overbreak or unauthorized increase in shaft penetration. However, in cases where a significantly increased quantity of concrete is anticipated due to existing ground conditions, such as losses into natural cavities in karstic rock, it is generally considered equitable to compensate the contractor for the additional volume of concrete placed beyond the plan dimensions of the shaft excavation (estimated from the diameter of drilling tools and casing); in this approach, the base unit cost includes a defined contractual overpour percentage, and any concrete placed beyond this base quantity is paid for separately as additional concrete. When such conditions are anticipated, these measurement and payment provisions provide an allowance in the unit price for drilled shaft concrete for a reasonable concrete overpour amount, but also provides compensation for unpredictable and unavoidable major concrete overpours.

Baseline Report: In particularly uncertain or variable ground conditions, consideration can be given to including a Geotechnical Baseline Report (GBR) as part of the contract documents. As discussed in Chapter 2, a GBR establishes the site conditions that can be expected and serves as the basis of the contractors' bid prices. The conditions defined in the report are then subsequently used to assess potential differing site conditions, and as a basis for determining payment. GBRs are well suited to design-build contracts for sites with highly variable subsurface conditions (Dwyre, et al., 2010). Guidance for the development and application of a GBR are presented by the American Society of Civil Engineers (ASCE, 2007). The GBR would be incorporated as part of the contract documents, and coordinated with the project specifications.

14.11 SUMMARY

A critical element of any drilled shaft project is the preparation of the technical specifications that appropriately address the specific conditions and requirements of the project. The guide specification presented in Appendix D can be used to identify the issues and provisions typically applicable to drilled shaft construction, and can be tailored to address project-specific requirements.

In addition, the information presented throughout this manual on the design and construction of drilled shafts provides valuable technical information that should be considered in the development of a construction specification for drilled shafts.

CHAPTER 15
INSPECTION AND RECORDS

Inspection and documentation of drilled shaft construction is a particularly important element of any drilled shaft foundation project. Unlike driven pile foundations where the driving resistance provides an indication of the nominal resistance of the pile, the successful installation of drilled shafts can only be assessed by verifying that the drilled shafts were installed in conformance with the contract documents and the approved Drilled Shaft Installation Plan, and in a manner consistent with the procedures used for the successful technique shafts and test shafts. Inspection is also needed to identify any unanticipated conditions encountered during construction that might jeopardize the performance or structural integrity of the completed shaft. In addition to observing drilled shaft construction, a complete written documentation of the work is essential for clearly communicating the drilled shaft construction activities to others, and to provide a permanent record of the work performed at each drilled shaft location. These records, coupled with post-construction integrity testing discussed in Chapter 16, also provide the primary means for identification and initial assessment of potential problems with a completed drilled shaft.

Thorough drilled shaft inspection records are also needed for assessing unanticipated ground conditions or ground behavior that may require an adjustment in shaft embedment or that might otherwise delay or add cost to the installation of the drilled shaft. In such cases, the inspector's records provide a basis for evaluating contractor claims of a "differing site condition" (DSC), as defined in the FHWA Geotechnical Guideline No. GT-15 (1996) and discussed in Section 2.6 of this manual, and for compensating the contractor for necessary additional work.

This chapter provides a general overview of the roles and responsibilities of personnel performing inspection of drilled shaft construction, and highlights specific issues to be addressed as part of the shaft inspection process. Also included are guidelines for preparing the necessary drilled shaft inspection records. For a more complete discussion of drilled shaft inspection procedures, the reader is referred to the training course for "Drilled Shaft Foundation Inspection" (NHI Course No. 132070) (2002) offered through the FHWA, and to the "Drilled Shaft Inspector's Manual" prepared jointly by the ADSC: The International Association of Foundation Drilling and the Deep Foundation Institute (2nd Edition, 2004). ADSC also provides informative videos demonstrating the various activities for inspecting the installation of drilled shafts, including the "Drilled Shaft Inspector's Video," "Construction and Inspection of Drilled Shafts Using the Slurry Method," "Concrete Placement" and "Safety in Foundation Drilling."

15.1 RESPONSIBILITIES

On a typical design-bid-build project, the drilled shaft inspector may be an employee of the owner or of a construction management firm engaged by the owner to provide construction inspection services. In general, the role of the inspector is to monitor the construction process so that the necessary records can be made, and to provide timely information to the foundation design engineer and/or the owner concerning unanticipated conditions and deviations from the drawings, specifications or the approved Drilled Shaft Installation Plan. The inspector should communicate and compare observations with the contractor, but the inspector does not direct the construction process; only the contractor has the authority to direct the construction operations and contractor personnel.

In a design-build contract, the drilled shaft inspector may be employed by the owner or a construction inspection firm hired by the owner, but more typically is a representative of an independent quality control organization that is hired by the design-builder. If the inspector is an employee of the design-builder's independent quality control group, the inspector will typically communicate with the designer-

of-record and construction staff of the design-builder, and with the quality assurance staff engaged by the owner. In essentially all other respects, the roles and responsibilities of the drilled shaft inspector will be the same for both design-bid-build and design-build type construction contracts.

Due to the specialized nature of drilled shaft construction and the particularly important role of the drilled shaft inspector for observing and documenting the work, the assigned drilled shaft inspector must be qualified by training and experience to perform this role. It is recommended that all drilled shaft inspectors successfully complete a drilled shaft inspector's training course, such as the one offered through the Federal Highway Administration (NHI Course No. 132070, 2002), and be certified for this work. In addition, the inspector should have prior experience as a drilled shaft inspector, or work under the supervision of an experienced drilled shaft inspector until the individual has obtained the necessary training and experience.

The drilled shaft inspector is typically a technician who has the appropriate training and experience for this work. A geotechnical engineer experienced with drilled shaft construction can also be a suitable drilled shaft inspector.

The inspector(s) should participate in pre-construction coordination meetings that address drilled shaft installation to better understand project requirements and the contractor's Drilled Shaft Installation Plan, as well as to better understand the particular concerns of the designer and the contractor. Pre-construction meetings are also useful for clearly reviewing the responsibilities of the drilled shaft inspector(s) and the procedures for preparing and submitting inspection forms, and for establishing the required lines of communication during construction.

At the beginning of any drilled shaft project, or when initially introduced to an on-going project, the drilled shaft inspector should meet with the foundation designer and/or geotechnical engineer for a briefing on the key issues and criteria applicable to that particular project. The inspector must also become familiar with all project documents related to the installation of the drilled shafts, as noted below.

Following are the specific roles and responsibilities of the drilled shaft inspector:

- Attend pre-construction meetings and construction meetings with the designer, the geotechnical engineer, the resident engineer and the contractor, as appropriate,
- Attend project safety training sessions and use necessary safety equipment,
- Be familiar with site conditions, including maintenance of traffic requirements, permit issues, utilities, etc.,
- Be familiar with general subsurface conditions, including types of soil and rock materials, and groundwater levels anticipated from the geotechnical exploration borings and available geotechnical reports,
- Be familiar with the relevant contract drawings, specifications, specification special provisions and payment provisions applicable to the drilled shaft foundations,
- Be familiar with the approved Drilled Shaft Installation Plan (Section 14.9), particularly information related to the procedures and equipment to be used, and verify that the available Plan is the most current,
- Be familiar with the drilled shaft installation criteria and tolerances,
- Have available all necessary inspection equipment, and verify the equipment is calibrated and in good operating condition (discussed in Section 15.2.1),
- Have available a sufficient number of blank forms to record the work (Section 15.4),
- Coordinate with the surveying team to obtain control elevations and confirm drilled shaft location,

- Observe and document all drilled shaft activities (Sections 15.2),
- Observe field testing of slurry (Section 15.2.3),
- Observe field testing of concrete, and verify concrete cylinder samples are properly obtained for laboratory testing (Section 15.2.6),
- Immediately inform the foundation design engineer, resident engineer and/or owner of any unanticipated conditions, problems or non-conforming work observed during drilled shaft installation,
- Communicate concerns and share inspection data with the contractor, and
- Measure and record the drilled shaft quantities used for documentation and payment purposes.

The inspector should have available in the field all relevant documents for the drilled shaft location under construction. Typical reference documents may include: the plan and details for the foundation unit; the specifications and specification special provisions for drilled shafts; the latest version of the Drilled Shaft Installation Plan; any additional instructions or installation criteria from the foundation design engineer and geotechnical engineer; and logs of the exploratory borings nearest to the foundation.

Prior to initiating drilled shaft construction, procedures should be established for the timely communication of information between the inspector and the foundation design engineer, geotechnical engineer, resident engineer, contractor and/or owner to quickly address any questions or problems that may arise. Timely resolution to construction issues is essential to avoid delay in the completion of the drilled shaft, and particularly to avoid constructing a defective or potentially deficient drilled shaft. It is the inspector's responsibility to immediately inform the design engineer, geotechnical engineer, resident engineer and/or owner of any unanticipated conditions, problems or non-conforming work, and it is the designer's and/or geotechnical engineer's responsibility to assess the information provided by the inspector, and to quickly respond to the contractor to help resolve the issue. Delays in the work need to be avoided or minimized not only because they impact the contractor's construction schedule but, more importantly, such delays may jeopardize the stability of the shaft excavation or the performance of the completed shaft.

The inspector's role extends beyond that of just an observer and recorder of construction activities; the inspector must also make judgments of the observed installation activities and communicate identified issues to the design engineer, geotechnical engineer and/or owner for quick resolution rather than delay action until after completion of the drilled shaft when the cost and schedule impacts associated with correcting a problem shaft will be considerably greater. With this objective in mind, it is appropriate for the inspector to share observations with the drilled shaft superintendent to allow the contractor to become aware of a potential shaft installation issue, and thereby provide the contractor opportunity in certain circumstances to address the issue directly, without delay. As noted previously, however, the inspector does not have authority to direct the contractor's operations in any way. And even if an observed condition is addressed by the contractor's field personnel, the inspector is still responsible for documenting the issue and communicating it to the design engineer, geotechnical engineer and/or owner for their consideration.

It is the responsibility of the contractor to determine the means and methods of drilled shaft construction suitable for the existing site conditions, provided they are in conformance with the requirements of the contract documents. The contractor is also responsible for installing the drilled shafts in accordance with the approved Drilled Shaft Installation Plan and in a manner consistent with the procedures used for the successful technique shafts and load test shafts. In addition, the contractor is responsible for constructing drilled shafts with the structural integrity necessary for their intended purpose.

15.2 INSPECTION ACTIVITIES

The inspector is responsible for observing all work performed during the various stages of drilled shaft installation, from initial set-up to completion of concrete placement, including removal of any temporary casing. Specific activities performed by the drilled shaft inspector during these various stages of drilled shaft construction are outlined below.

A sample checklist of drilled shaft inspector tasks is presented in **Table 15-1**. A comprehensive checklist similar to the one shown should be developed to incorporate the specific requirements for each particular project.

15.2.1 Set-Up

Following is a checklist of tasks to be performed by the inspector prior to the start of drilled shaft installation operations:

- Attend pre-construction meetings to become familiar with the planned drilled shaft construction procedures and installation criteria, and to understand the potential conditions and problems that may be encountered.
- Collect all project documents relevant to the specific drilled shaft being installed.
- Become thoroughly familiar with the project drawings and specifications for the drilled shaft foundations, and with the approved Drilled Shaft Installation Plan.
- Observe the existing conditions at the work site.
- Check all inspection equipment and verify they are in proper working condition and have been calibrated, where necessary. Inspection equipment will include measuring tape, weighted sounding tape, 4-ft long level, carpenter's square, slurry testing equipment (if necessary), concrete testing equipment, and concrete sample molds, among others.
- Collect the necessary inspection forms, and review with the geotechnical engineer the information to be recorded.
- Verify that the construction tools are in accordance with the equipment list presented in the approved Drilled Shaft Installation Plan.
- Check the dimensions of all drilling tools, and verify they are consistent with the design diameter of the drilled shaft and any rock socket.
- Check the dimensions of temporary and permanent casing (See Section 15.2.2).
- Check the steel reinforcing cage (See Section 15.2.5).
- Verify the protocol for communicating with the owner, design engineer, geotechnical engineer, resident engineer and contractor, and have available a list of contact phone numbers.
- Photograph drilling tools, casing, typical rebar cage and other equipment for further documentation and future reference.
- Use necessary safety equipment including hard hat, appropriate boots, safety glasses, safety harness, floatation vest for over-water work, etc.
- Attend the project safety training program, be familiar of safety procedures, and obtain contact phone numbers for use in emergency situations.

Table 15-1 SAMPLE DRILLED SHAFT INSPECTOR'S CHECKLIST

Contractor and Equipment Arrive on Site	**YES**	**NO**	**N/A**
1. Has the Contractor submitted a Drilled Shaft Installation Plan?			
2. Has the Drilled Shaft Installation Plan been approved?			
3. Does Contractor have an approved concrete mix design?			
4. Has Contractor run the required Trial Mix and slump loss test for the concrete mix design?			
5. If concrete placement is estimated to take over two hours, has Contractor performed a satisfactory slump loss test for the extended time period?			
6. If Contractor proposed a mineral or polymer slurry, do they have an approved Slurry Management Plan?			
7. Have you attended pre-construction conference with the Engineer and Contractor for clarification of drilled shaft installation procedures and requirements?			
8. Is Contractor prepared to take soil samples or rock cores on the bottom of the shaft, if required in the Contract Documents?			
9. Has the Contractor met the requirements for protection of existing structures?			
10. Has the site preparation been completed as specified?			
11. Does Contractor have all the equipment and tools shown in the Drilled Shaft Installation Plan?			
12. If casing is to be used, is it the right size?			
13. If Contractor plans to use a slurry, do they have the proper equipment to mix it?			
14. Is the manufacturer's representative on site at the start of slurry work?			
15. If a slurry de-sander is required, does Contractor have it on site and operational?			
16. Does Contractor's tremie meet the requirements of the Specifications?			
17. Do you have all the drilled shaft forms that are needed during shaft construction?			
Technique Shaft			
18. Is the technique shaft positioned at the approved location?			
19. Has Contractor installed the technique shaft as specified?			
20. Did Contractor cut off the shaft below grade as specified?			
21. Does Contractor have approval for revised procedures and equipment identified during technique shaft installation?			
Shaft Excavation and Cleaning			
22. Is the shaft being constructed in the correct location and within tolerances?			
23. Does Contractor have a benchmark for determination of the proper elevations?			
24. If core holes are required, has Contractor taken them in accordance with the Specifications?			
25. If a core hole was performed, was a Rock Core form completed and did Contractor maintain a log?			
26. If Contractor is using slurry, did they perform tests and report results in accordance with the Specifications?			
27. Is the slurry level being properly maintained at the specified level?			
28. Are the proper number and types of tests being performed on the slurry?			
29. Are you filling out the Drilled Shaft Excavation forms?			
30. If permanent casing is used, does it meet requirements of Contract Documents?			

31. If temporary casing is used, does it meet the requirements of the Specifications?			
32. Is the shaft within allowable vertical alignment tolerance?			
33. Is the shaft of proper depth?			
34. Does the shaft excavation time meet the specified time limit?			
35. If over-reaming is required, was it performed in accordance with Specifications?			
36. Does the shaft bottom condition meet the requirements of the Specification?			
Reinforcing Cage			
37. Is the rebar the correct sizes and configured in accordance with the project plans?			
38. Is the rebar properly tied in accordance with the Specifications?			
39. Does Contractor have proper spacers for the steel cage?			
40. Does Contractor have proper number and spacing of spacers for the steel cage?			
41. If steel cage was spliced, was it done in accordance with Contract Documents?			
42. Is the steel cage secured from settling and from floating?			
43. Is the top of the steel cage at proper elevation in accordance with specified tolerance?			
Concrete Placement			
44. Prior to concrete placement, has the slurry (both manufactured and natural) been tested in accordance with the Specifications?			
45. Was the tremie pipe within specified maximum height above the shaft base at the start of concrete placement?			
46. Was a flap valve or "pig" used to separate concrete from slurry at the start of concrete placement?			
47. Was the discharge end of the tremie maintained at the specified minimum embedment in the concrete			
48. If free-fall placement (dry shaft construction only), was concrete place in accordance with the Specifications?			
49. Did concrete placement occur within the specified time limit?			
50. Are you filling out the Concrete Placement and Volume forms?			
51. Did Contractor overflow the shaft until good concrete flowed at the top of shaft?			
52. If required, was the casing removed in accordance with the Specifications?			
53. Were concrete acceptance tests performed as required?			
Post Installation			
54. Is all casing removed to the proper elevation in accordance with Specifications?			
55. If required, has Contractor complied with requirements for Integrity Testing?			
56. Is the shaft within the applicable construction tolerances?			
57. Have all Drilled Shaft inspection forms been completed?			
58. Have you documented the pay items?			
Notes/Comments:			

15.2.2 Casing

The inspector should verify that the dimensions of temporary casing, including diameter, wall thickness and length, and any tip reinforcement or bands are consistent with the information presented in the approved Drilled Shaft Installation Plan. For permanent casing, the inspector should verify the casing dimensions and grade of steel are in accordance with the contract plans, and check that casing straightness and distortion to the cross-sectional shape are within specified tolerances. In addition, the inspector or other staff designated by the resident engineer should verify that the necessary inspection and weld testing of splices have been performed and documented.

As the casing is set in position, the inspector should check that the location and verticality of the casing are within the specified tolerances. Position can be measured by offset from established survey reference points. Verticality can be checked by use of a 4-ft long level, or by plumb bob.

The inspector should note the method used to advance the casing, and any difficulty encountered during casing installation including unexpected resistance, deformation, tilting or "walking" of the casing. The inspector should record the start and completion time for casing installation, and the cause, times and duration of any delays that occur, including stoppages for splicing the casing. At the completion of casing installation, the inspector should record the top and bottom elevations of the casing based on survey data provided by the contractor. For permanent casing, the casing elevations must be in accordance with the drawing elevations, within the specified tolerance. For temporary casing, the inspector should record the start and completion time for casing extraction, and the method used to remove the casing.

15.2.3 Drilling Fluid

The type of drilling fluid used, if any, should be in accordance with the approved Drilled Shaft Installation Plan. If slurry is used, the inspector should confirm that the proper slurry type is used and that it is mixed, placed and treated in accordance with the slurry manufacturer's recommendations, and as presented in the approved Drilled Shaft Installation Plan.

The inspector should observe slurry tests to verify they are preformed in accordance with the type of tests and frequency of testing detailed in the project specifications. Detailed procedures for performing tests on drilling slurry are described in the FHWA training course on Drilled Shaft Foundation Inspection (FHWA, 2002). These tests are summarized in Chapter 5 of this manual. Slurry tests are usually performed by the contractor and monitored by the inspector; however, the inspector should record all slurry test results and confirm that they are in accordance with the specified criteria, or in accordance with the slurry manufacturer's recommendations if not addressed in the project specifications. The performance of the slurry must be evaluated by the inspector during shaft excavation operations; if the slurry is inadequate for support of the excavation, or if the specified slurry properties cannot be achieved, the contractor is required to modify the slurry mix to achieve the specified performance. When testing is performed by the inspector, test results should be shared with the contractor for their use in adjusting the slurry mix.

The proper performance of the slurry is essential for the successful installation of drilled shafts by the wet method of construction. It is therefore particularly important that the inspector understand the use of slurry and the testing methods used for verifying compliance with specified slurry properties, as discussed in Chapter 5.

15.2.4 Drilled Shaft Excavation

Following is a checklist of tasks to be performed by the inspector during drilled shaft excavation operations:

- Verify the location of the top of the drilled shaft has been determined, and is within the specified tolerance.
- Note and document the excavation equipment used by the contractor, and verify it is consistent with the approved Drilled Shaft Installation Plan, note any changes to the drilling tools during shaft excavation, and note the depths of each change in tooling.
- Verify excavation procedures are in accordance with the requirements of the drawings and specifications, and consistent with the approved Drilled Shaft Installation Plan.
- Periodically check the verticality of the excavation by holding a 4-ft level on the Kelly bar, or by other suitable method.
- Sound the excavation depth at frequent intervals, and record the excavation elevation and corresponding time on the drilled shaft excavation log.
- If temporary casing is used, the bottom elevation of the casing and time of measurement should be recorded on the drilled shaft excavation log. Casing elevation should be recorded at the same times as the measurements for excavation elevation.
- If the dry method of construction is used, verify the temporary and/or permanent casing is properly sealed in an impermeable stratum based on observed seepage into the excavation.
- For the dry method of construction, verify the water infiltration rate and depth of water at the base of the shaft excavation are within the specified limits.
- For the wet method of construction, verify the slurry or water level is maintained at or above the specified minimum level at all times during and after shaft excavation.
- Perform or observe slurry testing, as discussed in Section 15.2.3 and Chapter 5.
- Observe the cuttings from the excavation, and record a description of these materials on the drilled shaft excavation log; correlate the materials encountered with the information shown in the logs for nearby subsurface investigation borings and previously installed drilled shafts. Note the elevation of each change in material encountered.
- Document the start and completion times, and equipment used for advancing the drilled shaft past obstructions.
- Document the level at which rock is encountered, assess the condition of the rock, and record any evidence of a sloping rock surface.
- When a probe hole below the bottom of the excavation is required by the specifications, verify the probe hole is performed and document the conditions encountered including material description, and drilling advance rate. It is usually preferable to perform the probe hole prior to the start of shaft excavation to avoid interruption of the work, but this is typically at the option of the contractor.
- The cross-section of the shaft is generally estimated based on the diameter of the drilling tools and casing used and the volume of concrete placed. However, estimating the drilled shaft diameter from concrete measurements is very imprecise. When a more accurate measurement of shaft cross-section is desired (such as for load test shafts or cases where cavities or voids are suspected), a borehole caliper can be used, Borehole caliper measurements are recommended for test shafts to better define the irregularity of the shaft cross section for use in evaluating the load test data. Borehole calipers can be either the mechanical type with outreaching arms to “feel” the side of the hole, or a sonic caliper which remotely senses the borehole wall from a suspended probe. Examples are illustrated in Figure 15-1.

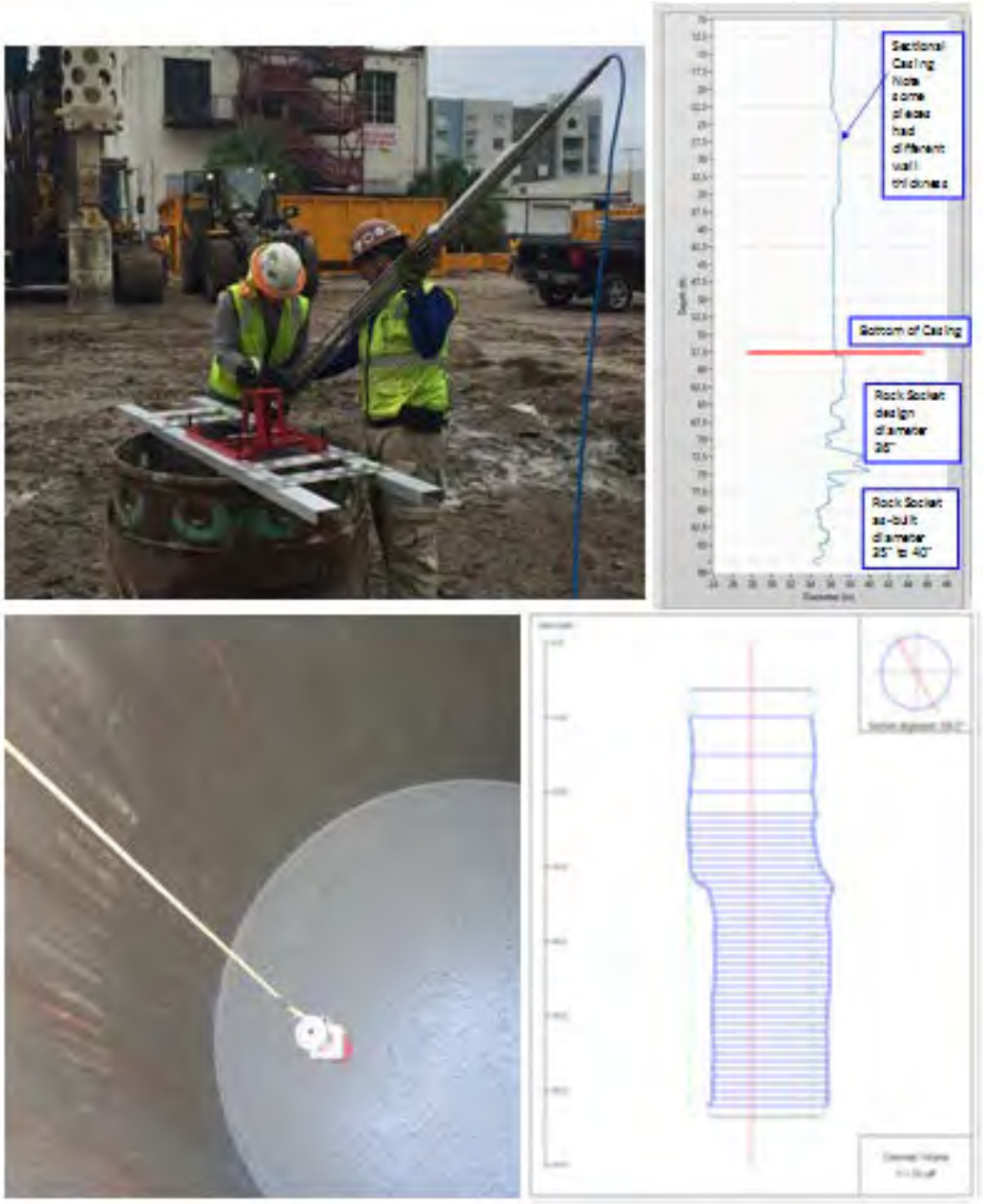


Figure 15-1 Example Borehole Calipers and Logs; Mechanical (top, photo courtesy Applied Foundation Testing, Inc.) and Sonic (bottom, graphic image courtesy Loadtest, Inc.)

- Document final bottom cleaning operations, including the type(s) of equipment used, the method used to check the bottom condition, and the results of the bottom inspection. The start and completion times for bottom cleaning operations, the time of each bottom inspection, and the observed bottom condition should be recorded on the shaft inspection logs. Bottom cleanliness can be assessed by the following methods:

a. A common method for assessing bottom cleanliness uses a weighted tape. Soundings are typically made at four sides of the shaft as well as the center to determine excavation depth and to assess the bottom condition. The criteria for acceptance may vary depending on the application, but generally the sounding is used to confirm that the base feels firm and that there is no soft, mushy material present. The weight shown in Figure 15-2 has a little foot on it, as might be commonly used for a soil bottom. If the base of the drilled shaft is on rock, the inspector may be looking for a sound "thump" of the bottom; a short piece of rebar, such as a #18 bar, is commonly attached to the bottom of the tape for this purpose.

b. Another, though uncommon method for sounding the bottom is with a neutral buoyancy rod.

c. For a visual assessment and digital documentation of bottom conditions, a video camera device such as shown in Figure 15-3 can be used. This method is often used for non-redundant drilled shafts and shafts relying on end bearing for a large portion of the geotechnical resistance. The bottom inspection tool shown in Figure 15-3a, called a Shaft Inspection Device (SID), is essentially a diving bell that utilizes compressed gas to displace the water or slurry for viewing the bottom in a wet excavation. The device is typically operated by a) setting the device on the bottom of the hole, typically at the center of the drilled shaft and at the four orthogonal sides of the shaft, b) pressurizing the gas to displace the slurry or water from the device, and c) viewing the bottom condition with the use of a video camera (Figure 15-3b) and one or more sediment measuring depth gauges mounted in view of the camera.

- Document any unusual or unanticipated conditions observed during drilled shaft excavation including, but not limited to, the presence of boulders or other obstruction; evidence of caving, bottom heave or other signs of hole instability; soil or rock conditions significantly different than those indicated in nearby investigation borings; slow or difficult drilling progress; loss of drilling fluid; surface settlement or other signs of ground loss around the shaft excavation; whenever the time of shaft excavation exceeds the specified time limit; or any other conditions that may potentially impact friction or end bearing resistance of the drilled shaft, as noted by the geotechnical engineer.
- Immediately notify the foundation design engineer or the geotechnical engineer of unusual or unanticipated conditions.



Figure 15-2 Weighted Tape for Sounding the Excavation Bottom



(a) (b)

Figure 15-3 Shaft Inspection Device for Visual Inspection of a Shaft Bottom in a Wet Excavation: (a) Device lowered into a 60-inch diameter shaft, and (b) Camera shot of excavation bottom with three feeler gauges spaced at 120 degrees (photo courtesy of Applied Foundation Testing, Inc.).

Current practice for inspection of the base of the shaft excavation typically relies on inspection made from the surface. It is recommended that inspection personnel not enter the drilled shaft unless it is absolutely necessary, and then only if all prudent and required safety precautions are taken.

The foundation design engineer or geotechnical engineer should review each of the drilled shaft excavation logs to verify that the work was performed in general accordance with the contract documents and with the approved Drilled Shaft Installation Plan, and is compatible with the friction and end bearing resistance requirements for the shaft.

15.2.5 Placement of Reinforcement

In most cases, the steel reinforcement is delivered to the project site as individual bars in predetermined lengths and shapes for fabrication into rebar cages at the job site. Alternatively, pre-assembled cages may be delivered to the job site. Inspection of the reinforcement includes the following tasks:

- If steel coupon testing is required, the inspector or other personnel designated by the construction manager should verify that the necessary coupon samples have been taken for testing by others.
- Verify that the individual bars and the fabricated rebar cage are supported off the ground, and protected against contamination by soil, grease and other deleterious material.
- Check the rebar cages after they are fabricated to verify that they are in agreement with the steel grade, dimensions, arrangement, sizes and spacing shown on the design drawings or approved shop drawings, and in accordance with the specifications.
- Verify that intersecting bars of the rebar cage have been suitably tied with wire in accordance with the contract requirements or the approved Drilled Shaft Installation Plan.
- Verify that centering devices are installed in accordance with the types, sizes and spacing specified and shown in the approved Drilled Shaft Installation Plan. The inspector should also verify rebar cage bottom spacers (“boots”) are installed, if required.

- Verify that CSL tubing is installed in accordance with the contract specifications and the approved Drilled Shaft Installation Plan. The inspector should verify that the tubes are of the specified material; are undamaged and straight; have the required length and extend to the specified bottom level; have suitable watertight couplers and end caps; and are securely attached to the rebar cage in the require number and arrangement.
- Verify that other embedded devices, including shaft base post-grouting systems, TIP cables, bi-directional load test devices, and instrumentation have been installed, where required. Typically, a specialty subconsultant will be responsible for detailed inspection and verification checks of these atypical embedment items.
- Observe the lifting of the rebar cage to determine if there is any damage or permanent distortion to the rebar cage, CSL tubes, or any embedded items due to the lifting operation. Note any excessive deformation of the rebar cage during lifting.
- If the cages are spliced as they are lowered into the drilled shaft excavation, the inspector must verify that the splices are performed in accordance with the requirements of the drawings, specifications, and approved Drilled Shaft Installation Plan, particularly noting splice lap lengths; the type, size and installation procedure for mechanical splices; and the staggering of the splices.
- Confirm that temporary internal stiffeners are removed from the cage as it is being lowered into the shaft excavation.
- Verify that any missing or damaged centering devices are replaced as the cage is lowered into the shaft excavation.

The inspector should be alert to cage installation procedures that may increase the risk of soil or rock material being dislodged from the side of the shaft excavation as the cage is lowered into the hole. Procedures that may pose such a condition include: rapid lowering of the rebar cage; not maintaining the cage concentric with the drilled shaft excavation; missing or damaged centering devices; and protrusion of rebar steel or other embedded items beyond the outside diameter of the cage. Following installation of the rebar cage, the inspector should sound the bottom of the shaft excavation, when possible, to assess the presence of debris on the bottom. If there is a noticeable change from the condition or elevation determined just prior to placement of the reinforcement cage, the inspector should immediately inform the foundation design engineer. In certain cases the engineer may not accept such a condition, and may direct the contractor to remove the reinforcing cage and re-clean the bottom of the shaft.

15.2.6 Concrete Placement

Proper concrete placement is essential to a successful drilled shaft installation free of defects that may jeopardize the structural integrity of the completed shaft. Accordingly, it is particularly important that the concrete placement operations be carefully inspected and documented. Thorough inspection and logging of concrete placement operations are necessary not only to document the work performed, but also to provide the information necessary for assessing possible anomalies identified in subsequent integrity testing of the completed shaft, as discussed in Chapter 16.

The inspection of concrete placement operations involves two separate activities including a) concrete sampling and testing, and b) inspection of concrete placement into the shaft excavation. Concrete sampling and testing is essential to verify that the required concrete mix is delivered to the job site and that the concrete has the workability properties necessary for proper placement, particularly for projects requiring the wet method of construction. Equally important is the inspection of concrete placement into the shaft since improper placement could result in major defects and rejection of the completed shaft. Following are inspector checklists for each of these inspection activities. Further discussion and details of

concrete sampling and testing are described in the FHWA training course on Drilled Shaft Foundation Inspection (FHWA, 2002) and in Section 7.7 of this manual.

Concrete Sampling and Testing

- Obtain the delivery ticket for each load of concrete delivered to verify that the proper concrete mix has been furnished, and to determine the volume of concrete in the load.
- Verify the mix is appropriate for the ambient temperature conditions at the time of placement. (Since concrete slump loss rate is highly influenced by the ambient air temperature, it is important the mix delivered to the job site is one that has been tested and approved at an ambient temperature equal to or greater than the ambient temperature at the time of placement. This issue is especially important for deep, large diameter drilled shafts installed using the wet method of construction that may require an unusually long concrete placement time.)
- Check the batch time to verify that the age of the concrete is within the specified time range.
- Verify that routine slump or slump flow tests, concrete temperature testing, and air entrainment testing, if required, are performed at the frequency noted in the specifications or directed by the engineer. Slump loss tests (slump tests on the same concrete sample at selected time intervals) are occasionally performed at the placement site using concrete obtained from the initial load delivered to the shaft; such testing provides another means of assessing the workability of the mix over time, particularly for drilled shafts with anticipated long concrete placement time or shafts poured during periods of high ambient air temperature conditions.
- Verify that concrete cylinder samples are prepared at the frequency noted in the specifications or directed by the engineer. Verify samples are stored in an approved manner until time for testing.
- Look for any conditions that may adversely impact concrete placement, including segregation of the aggregate, clumping, insufficient slump, etc.

Concrete Placement

- Check the rate of water infiltration into the shaft excavation and the height of water at the bottom of the shaft. If either condition exceeds the specified limits, verify the contractor fills the shaft excavation with water or slurry, as appropriate, and places the concrete by the wet method.
- Verify the diameter of the tremie pipe or pump line meets the minimum specified diameter.
- Verify the tremie pipe or pump line has sufficient length to reach the bottom of the shaft excavation.
- Verify the inside and outside surfaces of the tremie pipe or pump line are clean and free of protrusions that may impede concrete flow or extraction of the tremie or pump line from the shaft.
- Verify the tremie pipe or pump line and all joints are watertight.
- Verify that the distance above the bottom of the tremie or pump line is marked and numbered at intervals of not more than 5 feet to allow determination of the depth to the bottom of the tremie or pump line during concrete placement.
- Verify all CSL tubes are filled with water and provided with a removable top cap prior to the start of concrete placement.
- Verify a bottom flap valve is used to prevent water or slurry entering the pipe prior to placing the initial charge of concrete into the tremie or pump line. Alternatively, verify a “pig” of the proper size and material is placed at the top of the pipe to separate the water or slurry from the initial charge of concrete.
- In wet excavations, verify the bottom of the tremie or pump line is within the specified maximum height above the bottom of the shaft at the start of concrete placement.
- Verify the bottom of the tremie or pump line remains embedded in the concrete the minimum specified depth for the entire placement period.
- In wet excavations, verify the fluid pressure within the casing (drilled fluid and concrete) exceed the external hydrostatic pressure at all times during concrete placement.

- If temporary casing is used, verify the casing is slowly extracted to allow the concrete to flow into the space vacated by the casing.
- If temporary casing is used, verify the top of concrete level remains within the casing until completion of concrete placement.
- If temporary casing is used, verify the casing is pulled vertically out of the shaft to avoid disturbance to the soils around the shaft.
- If the dry method of construction is used, verify that the free-fall height of the concrete drop does not exceed the maximum specified height.
- If the dry method of construction is used, verify the concrete stream does not impact the rebar cage or the side of the shaft excavation.
- Maintain a log of concrete placement as described in Section 15.4.
- During concrete placement, maintain a plot of actual and theoretical concrete volume versus elevation, as described and illustrated below, to assess possible bulging, necking and instability of the shaft.
- Verify the concrete is overpoured until concrete of good quality reaches the top of the shaft. Alternatively, if the contractor elects to chip away any contaminated concrete at the top of the shaft, verify the concrete level extends above the shaft cutoff elevation the minimum amount noted in the approved Drilled Shaft Installation Plan to provide good quality concrete at and below the design cutoff elevation.
- For drilled shafts constructed by the wet method, verify the total concrete placement time, measured from the batching of the initial load of concrete placed in the shaft until the end of concrete placement and removal of any temporary casing, does not exceed the approved maximum placement time for the mix, as determined from slump loss testing of the mix used.
- Immediately inform the foundation design engineer or geotechnical engineer of any unusual observations noted during concrete placement operations.

During concrete placement, it is important to measure and maintain records of concrete volume placed as a function of elevation in the shaft. It is particularly important to document concrete volume in a load test shaft since any significant deviation from the theoretical volume may have an influence on the load test results. The top of concrete elevation during construction can typically be determined using a simple weighted tape. When compared to the theoretical volume required to fill the hole, these measurements can help to identify areas where overbreak may have occurred or where concrete may be filling voids, such as solution cavities in karstic limestone formations. A sudden rise in measured concrete elevation in comparison to the incremental volume of concrete placed may be evidence of a cave-in during concrete placement. Such unusual measurements during concrete placement may warrant further investigation. An example of measured and theoretical concrete volume versus depth is illustrated in Figure 15-4.

When temporary casing is removed during concrete placement, a measurement of concrete level should be obtained just prior and immediately following removal of each casing section. A comparison should then be made of the measured concrete drop with the theoretical volume of concrete needed to replace the volume of steel removed from the hole. This comparison is useful in identifying any void space that may have been present outside the casing.

The most common method for estimating the volume of concrete placed is to record the elevation of the concrete level in the shaft after each truck is discharged, with the volume determined by the truck tickets. The volume of concrete within the tremie or pump lines must be estimated and subtracted from the total. In some cases, a stroke counter on a pump may be used to estimate the volume pumped, but the volume per pump stroke must be determined by calibration. If a bucket with a known volume is used to place the concrete, an approximate volume of concrete placed can be determined by bucket count.

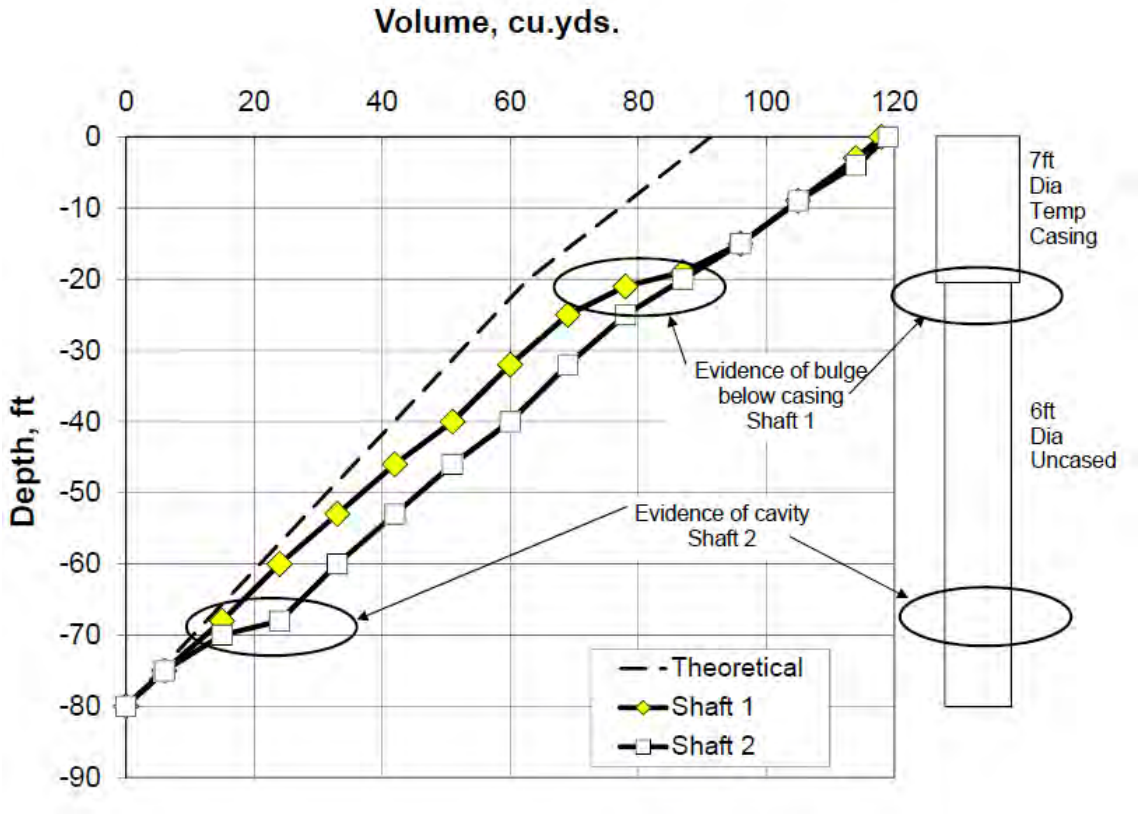


Figure 15-4 **Example Concrete Volume Plots**

15.2.7 Completed Drilled Shaft

At the completion of each drilled shaft installation the inspector should perform the following tasks:

- Obtain from the contractor the surveyed final, as-built location of the drilled shaft and the actual top of concrete elevation.
- Complete a summary installation form that documents the as-built dimensions and elevations of the drilled shaft, as discussed in Section 15.4.
- If paymemt for drilled shafts is based on unit pricing, determine the quantity for each pay item at each completed drilled shaft and verify agreement of the contractor for these quantities.
- Compile and file all inspection records for the completed shaft.
- Update any project control summary tables and charts for drilled shaft construction.

15.3 COMMON PROBLEMS

The vast majority of drilled shafts are constructed without problems related to loss of geotechnical resistance within the bearing strata, or to a defect that compromises the structural integrity of the shaft. When problems occur, they are likely to fall into one of the categories listed in Table 15-2. The inspector should discuss potential problems with the foundation design engineer or project geotechnical engineer for a better understanding of the risk of these and other types of problems for the anticipated project conditions and selected method of construction. The inspector should then be alert to the construction operations that can produce these problems.

TABLE 15-2 COMMON PROBLEMS DURING DRILLED SHAFT CONSTRUCTION (AFTER BAKER ET AL., 1993; O'NEILL, 1991)

Type of Problem	Potential Cause of Problem
Shaft off location or out of plumb	Improper set-up or poor alignment while drilling
Shaft with insufficient embedment in proper bearing stratum.	Bearing stratum misidentified or length not properly measured
Crack in shaft	Shaft hit by construction equipment early in curing process
Bulge or neck in the shaft	Soft ground zones that were not cased
Caving of shaft wall	Improper use of casing or slurry; failure to use weighting agent with slurry; casing not sealed in a stable stratum
Reduction in side resistance due to excessive mudcake buildup	Failure to agitate slurry or to place concrete in a timely manner
Temporary casing cannot be removed	Crane and/or vibratory hammer for extracting casing ineffective in squeezing ground; large set-up of soil friction after installing casing; casing wedged in rock or by boulder
Horizontal separation or severe necking of shaft	Pulling temporary casing with concrete adhering to it; inadequate penetration of casing into concrete during casing extraction, or casing pulled out of concrete
Horizontal sand lens in concrete	Tremie or pump line pulled out of concrete in wet hole; insufficient workability for shafts requiring long concrete placement duration; insufficient head within casing when raising casing
Soil intrusion on the side of the shaft	Use of telescoping casing where concrete from inner casing spills into annular void behind the outer casing; low concrete slump; reinforcing bars too closely spaced
Soft shaft bottom or CSL anomaly at/near bottom of shaft	Incomplete bottom cleaning, side sloughing, or sedimentation of cuttings from slurry column
Voids outside of cage	Low concrete slump, aggregate too large, and/or reinforcing bars too closely spaced
Concrete defects	Tremie pipe joints not watertight; tremie/pump line not at bottom of shaft at start of concrete placement; tremie/pump line raised above concrete level; concrete flow into annular void between temporary and permanent casings; concrete slump inadequate for duration of concrete placement; excessive sediment in slurry
Honeycombing, washout of fines or water channels in the concrete	Concrete placed directly into water; excessive groundwater head; excessive bleed water in concrete mix
Folded-in debris	Insufficient cleaning of shaft base; excessive sand content in slurry
Clogged tremie or pump line	Concrete with insufficient slump or slump retention; interior of pipe not clean; segregation of concrete aggregates
Rebar cage lifted during concrete placement	Weight of rebar cage insufficient for rising concrete; tremie/pump line embedded too deep in concrete; rebar cage caught on tremie/pump line; concrete arch between casing/cage
Rebar cage settles during concrete placement	Missing or inadequate number/spacing of rebar cage spacers; insufficient support of cage at bottom of shaft excavation; insufficient cage stiffness

15.4 RECORDS AND FORMS

Contracting agencies typically have standard forms that they use for documenting drilled shaft construction. Such forms may include a checklist for rebar cage fabrication; a fabrication checklist for permanent casing; a shaft excavation log; a shaft excavation bottom inspection form; and concrete placement forms. When contracting agencies develop their own standard inspection logs, they are often attached to the drilled shaft specification section and their use may be a contract requirement. In the absence of standard forms from the contracting agency, the sample inspection forms included in Appendix E may be used for documenting the various drilled shaft construction activities.

The attached sample inspection forms each identify the specific items to be recorded. Much of the specific information to be entered on these forms is self-evident, or has been noted in the preceding sections of this chapter. Sample completed forms are also included in Appendix E to illustrate the type of information to be recorded. Following, however, is a brief discussion regarding general procedures for using these forms.

- Each page of the log should identify the project name, project location, structure identification (for projects with multiple structures), shaft identification number (corresponding to the number shown on the design drawings or working drawings), date, and page number.
- Each page of the log should record the name(s) of the inspector(s), and note the time of any change of inspector during the course of drilled shaft installation.
- Data recorded on the inspection logs should be referenced to elevations rather than depth. The use of elevations eliminates any uncertainty regarding the exact level of the noted item, and avoids confusion resulting from a change in the reference level during shaft installation. Recording data in terms of depth should generally be avoided, and used only when the elevation of the reference level is determined by survey and clearly documented on the inspection forms.
- The date and time for the start and completion of each activity, and the beginning and end of any interruptions to the work, need to be recorded on the inspection forms. When repetitive operations are performed, such as the repeated insertion of excavation tools, it is useful to record the time for each step, i.e. the time for each insertion of the drilling tool, and the time when it is removed from the hole, along with periodic measurement of excavation level. To avoid potential confusion, it is preferred to use a 24-hour (military) clock reading rather than a 12-hour clock reading.

In addition to the inspection forms described above, each inspector should maintain a daily report to record information and construction activities that are normally not entered on the shaft inspection forms. Following is a sample list of just some of the items that might be recorded in the inspector's daily report:

- Weather conditions, river level, etc.
- Equipment and work force on site.
- Equipment repair or maintenance performed.
- General construction activities not directly related to drilled shaft installation, e.g. mobilizing to a new work area, setting up templates, site excavation or filling, etc.
- Delays to the work, e.g. equipment breakdown, severe weather conditions, change in work schedule, etc.
- Tracking of time and material for unanticipated tasks, such as penetrating obstructions.
- Information obtained from the contractor or the engineer.
- Instructions given to the work crew by the contractor's superintendent.
- Any time spent on other activities not related to drilled shaft construction.

All records must be collected, organized and maintained in a central file in accordance with the document control procedures established for the project. Copies of the drilled shaft inspection logs should be distributed to resident engineer and the foundation design engineer (or project geotechnical engineer) in a timely fashion for review and evaluation.

15.5 SUMMARY

This chapter discussed the responsibilities of the drilled shaft inspector and the varied tasks required for monitoring and documenting the drilled shaft installation. Sample checklists and record keeping forms were also presented illustrating the specific information to be collected for each step in the installation of the drilled shafts.

It was emphasized that the inspector is not just an observer and recorder of the drilled shaft construction activities, but must be proactive in identifying issues and problems that may impact the performance of the completed drilled shaft, and must communicate these observations in a timely manner to the resident engineer, the foundation design engineer and the project geotechnical engineer for evaluation and resolution. It was also emphasized that the inspector does not have the authority to direct the contractor's operation in any way, but should share observations with the contractor's superintendent to allow the contractor the opportunity to make their own assessment and to take action they may consider appropriate to address the identified issue while it is still possible to do so.

As discussed in this chapter, the records collected by the inspector are a fundamentally important element of any drilled shaft project since they serve as the only reliable information that can be used to verify that the drilled shafts were installed in accordance with the requirements of the contract documents and the approved Drilled Shaft Installation Plan, and also serve as the principal means for evaluating the adequacy of the completed drilled shaft. In addition, the inspector's records provide a basis for evaluating contractor claims of a "differing site condition," and for compensating the contractor for necessary additional work.

CHAPTER 16
INTEGRITY TESTING

16.1 INTRODUCTION

Post-construction integrity testing plays an integral role in the drilled shaft design and construction process for transportation infrastructure. The most common purpose of post-construction testing is quality assurance of concrete placement, in which some characteristic of the hardened concrete is measured to assess its integrity. Most tests used for this purpose have no permanent effect on a drilled shaft and are therefore referred to as "non-destructive integrity tests," or NDT. The interpretation and use of NDT for drilled shaft assessment is referred to as "non-destructive evaluation" or NDE. In combination with quality construction and inspection practices, NDE provides a tool for ensuring the as-built foundation satisfies the construction specifications and confidence that it will perform as assumed in the design. A major point of emphasis in this manual is that NDE must be viewed within the context of a well-designed and executed inspection process and not as a substitute for quality construction or inspection.

A second application of the tests described in this chapter is to evaluate drilled shafts when there is reason to suspect that a defect exists. Most drilled shafts are constructed routinely, without difficulty, and are sound structural elements. On occasion, however, there is reason to question the integrity of a drilled shaft based on construction observations, anomalous results of concrete cylinder tests, or unexpected performance during construction of the superstructure (*e.g.*, excess settlement). In these cases, the objective of testing is to define the potential problem and obtain information that can be used to determine if a remediation plan is needed. These tests may include both non-destructive and destructive (*e.g.*, concrete coring) methods.

From a management perspective, post-construction tests on completed drilled shafts can be placed into two categories:

- Planned tests that are included as a part of the quality assurance procedure, and
- Unplanned tests that are performed as part of a forensic investigation in response to observations made by an inspector or contractor, or from concrete cylinder testing, that indicate a defect might exist within a shaft.

Planned tests for quality assurance typically are non-destructive and are relatively inexpensive. Such tests are performed routinely on drilled shafts for transportation projects in the United States. Unplanned tests performed as part of a forensic investigation will normally be more time-consuming and expensive, and the results typically require more extensive evaluation than those of planned tests performed properly.

The following definitions from ASTM D6760 are presented to clarify several terms used in this chapter and discussed further in Chapter 17:

- Anomaly, n. - an irregularity or series of irregularities observed in the NDT results, indicating a possible flaw.
- Flaw, n. – any deviation from the planned shape or material characteristics (or both) of the drilled shaft
- Defect, n. - a flaw that, because of either size or location, may significantly detract from the drilled shaft's performance

16.2 NON-DESTRUCTIVE INTEGRITY TESTS

The most common NDT methods used in the transportation sector are summarized in Table 16-1. This section provides an overview of the tests listed in Table 16-1, as well as other less common tests. These tests generally require expert knowledge for performance and interpretation. Technician-level expertise is required for conducting the field tests, while interpretation of results should be done by a qualified engineer in consultation with the project geotechnical engineer. Most of these tests also require specialized software for data acquisition and processing. When a transportation agency employs one or more of these tests, it should do so either with its own employees who have been trained in the performance and interpretation of the test, or it should employ a qualified outside firm. It is also common in U.S. practice for NDT/NDE to be included in the construction contract, in which case the contractor is responsible for hiring a qualified firm to conduct and interpret the tests. While this approach often works well, it can raise difficult questions when a dispute arises over anomalous readings and their interpretation. If the NDE firm is under contract to the foundation contractor, it is not clear whether they are representing the interests of the contractor or the transportation agency.

TABLE 16-1 COMMON NDT METHODS FOR DRILLED SHAFTS IN TRANSPORTATION APPLICATIONS

Test Feature:	Crosshole Sonic Logging (CSL)	Thermal Integrity Profiling (TIP)	Gamma-Gamma
ASTM or Other Standard	ASTM D 6760	ASTM D 7949	Caltrans Test 233
Basic Concept	Acoustic signals generated in embedded access tubes are measured in adjacent tubes; Signal velocity and strength provide evaluation of concrete quality between the tubes	By monitoring the temperature profile generated by the heat of hydration as concrete cures, shaft geometry can be inferred; low temperature zones indicate potential defects; Requires temperature-measuring cables or access tubes embedded in the concrete	Gamma rays emitted from a source are backscattered by concrete and measured by a detector; measured gamma ray counts correlate to concrete density; source and detector are located in a single probe lowered into access tubes
Primary Application	Assessment of concrete quality inside the reinforcing cage	Assessment of concrete quality for the entire cross section	Assessment of concrete quality around the perimeter of the shaft
Limitations	Difficulty locating defects outside the line of sight between tubes; Tubes must be installed with the cage prior to concrete placement	May be less effective in detecting anomalies at the tip; Requires thermal wires or access tubes to be cast into the drilled shaft; Limited time window for obtaining measurements	Anomaly detection is limited to a zone of several inches from the tubes located just inside the rebar cage; Tubes must be installed prior to concrete placement; Requires proper management to handle, transport, and store radioactive materials
Advantages	Relatively accurate and relatively low cost; Effective for identifying anomalies in the core concrete	Relatively accurate, low cost, and fast; capable of capturing anomalies anywhere in the cross section	Relatively accurate and relatively low cost; Highly effective in identifying anomalies around the shaft perimeter
Variations and Related Tests	Cross-Hole Tomography; Perimeter Sonic Logging; Single Hole Sonic Logging		

16.2.1 Sonic Methods

Crosshole Sonic Logging: The Crosshole Sonic Logging (CSL) method is the most widely used test for quality assurance of drilled shaft concrete. A schematic of the method is shown in Figure 16-1. Vertical access tubes are cast into the shaft during construction. The tubes are normally placed inside the rebar cage and must be filled with water to facilitate the transmission of high frequency compressional sonic waves. Typically, one tube is installed for each foot of shaft diameter, and the tubes extend from the base of the shaft to several feet above cut-off level. An acoustic transmitter (T) is lowered to the bottom of one access tube and a receiver probe (R) is lowered to the same depth in one of the other tubes. The transmitter emits an acoustic impulse at an assigned frequency, usually 30 to 50 kHz. The signal travels through the concrete and is picked up by the receiver. The probe cables are pulled upward simultaneously such that the transmitter and receiver probes are always at the same elevation (zero-offset). Logging involves measuring and recording the emitted and received signals at specified increments of depth (typically 2 inches). The ability to obtain acoustic profiles between multiple pairs of tubes makes it possible to characterize the position of an anomaly in the shaft. Analysis of multiple profiles can also provide an idea of the size of a potential defect. These features, combined with low to moderate cost, are the primary advantages of CSL testing.

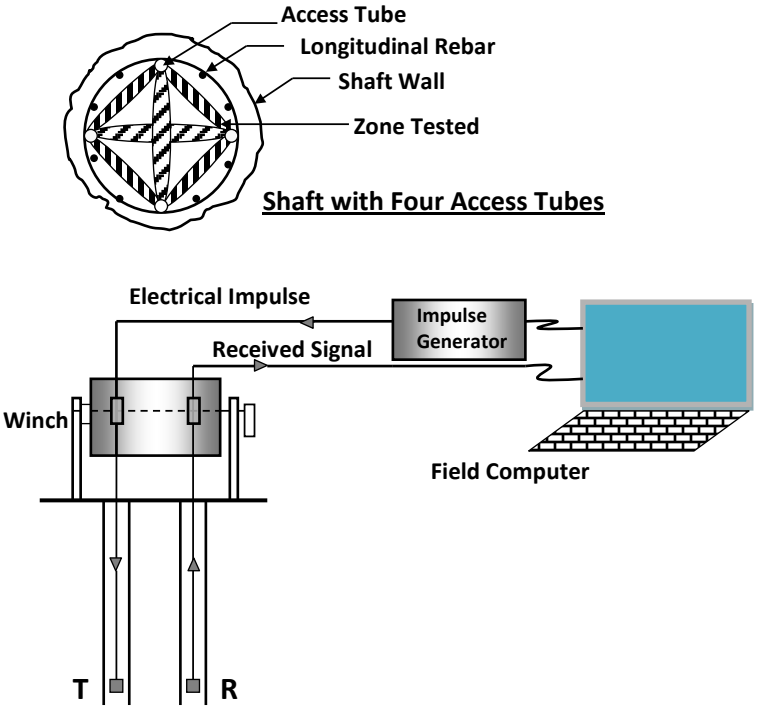


Figure 16-1 Diagram of Crosshole Acoustic Logging System

Integrity of the concrete between source and receiver is evaluated on the basis of two test results: signal travel time and signal strength. Travel time can also be converted to velocity (V), given by V = (d/t), where d = center to center spacing between tubes, and t = measured travel time. Travel time is taken as the time required for the leading edge of the ultrasonic signal to travel from the transmitter to the receiver, referred to as First Arrival Time, or FAT. Signal strength is typically evaluated in terms of relative energy (RE), obtained by integrating the absolute value of the signal amplitude for a defined time period, and expressed in units of decibals (dB). Strong anomalies (sharp increase in FAT or corresponding decrease in velocity), combined with decreased signal strength (decrease in RE), are interpreted as potential defects (flaws).

The computed velocity of the acoustic signal can be compared to the theoretical velocity of a compressional wave through concrete, given by:

$$V_c = \sqrt{\frac{\alpha E}{\rho}} \qquad 16\text{-}1$$

$$\alpha = \frac{1-\nu}{(1+\nu)\ (1-2\nu)} \qquad 16\text{-}2$$

$$\rho = \frac{\gamma}{g} \qquad 16\text{-}3$$

where:

V_c = theoretical compressional wave velocity in concrete
E = modulus of elasticity
ν = Poissons ratio
ρ = mass density of concrete (mass/unit volume)
γ = unit weight of concrete, and
G = acceleration due to gravity (32.2 ft/sec^2)

Using properties of concrete with compressive strengths in the typical range of 3,000 to 5,000 psi, Equation 16-1 yields V_c = 10,000 to 11,500 ft/second, respectively. However, velocity is also affected by frequency, and experience demonstrates that normal quality concrete will exhibit sonic velocities closer to an average of 13,000 ft/sec, a value sometimes used as a baseline against which CSL measured velocities are evaluated. More typically, signal processing software is used which computes a running average of velocity over a specified depth interval (typically 10 to 12 ft) and compares individual readings to this baseline. The degree to which the measured velocity deviates from the baseline value can be given in terms of a velocity reduction (VR), expressed as a percentage:

$$VR = \left(1 - \frac{V}{V_b}\right)100\% \quad \approx \left(1 - \frac{V}{13{,}000}\right)100\% \qquad 16\text{-}4$$

in which V_b = baseline velocity (assumed to be 13,000 ft/sec above for illustrative purposes). In contrast to concrete, the sonic velocity in water is approximately 5,000 ft/sec and in air is approximately 1,000 ft/sec. A qualitative rating of the concrete condition, based on VR% and the degree of energy reduction in the received signal, is given in TABLE 16-2. This rating scheme is widely used by many state and federal transportation agencies as a basis for initial assessment of CSL test results.

More recently, the Deep Foundations Institute (DFI, 2018) has proposed a revised set of guidelines for evaluation of drilled shaft CSL test results. The DFI guidelines are considered preliminary and are not recommended to replace the guidelines in Table 16-2 at the present time. In the proposed DFI rating, a running average of FAT measurements is made over a specified depth interval (typically 10 to 12 ft) and taken as the baseline value against which each individual FAT reading is compared. The degree to which the measured FAT deviates from the baseline is expressed in terms of a percentage increase. CSL results are then classified on the basis of percentage FAT increase and energy reduction. Refer to DFI for further details.

TABLE 16-2 CONCRETE CONDITION RATING CRITERIA

Velocity Reduction, VR (%)	Signal Distortion/Strength	Concrete Rating	Indicated Conditions
0 – 10	none/normal energy reduction ≤ 6 dB	Good (G)	Acceptable quality concrete
10 – 20	minor/lower energy reduction 6.1 to 9 dB	Questionable (Q)	Minor contamination, intrusion, or questionable quality concrete
> 20	severe/much lower energy reduction > 9 dB	Poor/defect (P/D)	contamination, intrusion, and/or poor quality concrete
No signal	None	No Signal (NS)	Intrusion or severe defect; could also be caused by tube debonding
≈60	severe/much lower energy reduction ≥ 12 dB	Water (W)	water intrusion or water-filled gravel intrusion with few or no fines

Figure 16-2 shows an example CSL profile based on measurements made in a single tube combination. The left side of the figure shows the computed wave velocity (heavy line) and the relative energy (thin line) plotted on a log-energy graph, with lower values to the right. The right side of Figure 16-2 is a 'waterfall diagram,' obtained by nesting of signal arrival times versus depth measurements. ASTM D 6760 recommends the waterfall diagram be included with the reported CSL test results. The left edge of the waterfall diagram represents the first arrival time, or FAT, and depths of low intensity, such as at depths of 1, 7 and 10.5 m (3, 22 and 35 ft), indicate decreased signal strength. The FAT increases (velocity decreases) at the three depths noted above are 40, 38, and 14 percent, respectively, and the corresponding energy reductions are 7.3, 9.4, and 7.5 dB. The anomalies at depths of 1 m and 7 m would place this drilled shaft in the Poor/defect rating of Table 16-2 and Class C of Table 16-3.

The size of anomaly or defect that can be detected by CSL decreases with increasing frequency of the emitted signal. Theoretically, the smallest defect that can be reasonably detected is about one-fourth of the wave length of the transmitted acoustic signal. However, experimental studies on full-scale drilled shafts suggest that additional factors limit the size of a defect that can be detected by CSL. Amir and Amir (2009), based on a review of the literature and on field testing of a drilled shaft with known inclusions, concluded that a defect located halfway between two access tubes is detectable only if its size exceeds about one third of the tube spacing or about 10 percent of the shaft cross section. They also note that an anomalous signal created by a defect depends not only on the size of the defect, but also on its location. The closer a defect is to an access tube, the larger it appears in 2-D or 3-D tomography (described below).

Access tubes for CSL typically are Schedule 40 steel or PVC with a 1.5 to 2-inch inner diameter to accommodate the probes. The tubes are attached to the inside of the rebar cage and normally extend the full length of the drilled shaft. The tubes are plugged on their lower ends to keep out concrete. A rule of thumb is to place the access tubes uniformly around the cage, using one longitudinal access tube for each foot of shaft diameter. At least two tubes are installed. It is important for the tubes to be vertical and at a constant spacing over their entire length so that measured differences in travel time of sonic signals do not occur as a result of differences in tube spacing, which could lead to incorrect interpretation of test results.

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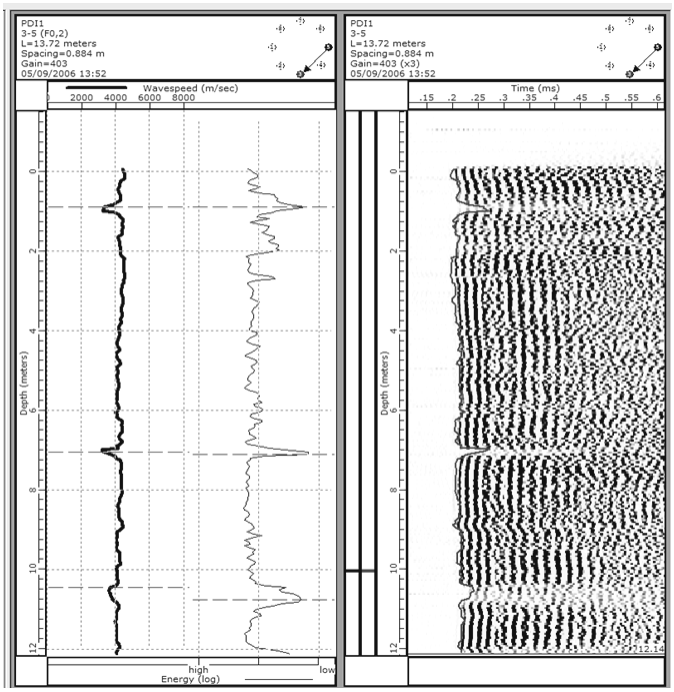


Figure 16-2 Crosshole Sonic Log for a Drilled Shaft with Known Inclusions (Likens et al., 2007) (1 m = 3.281 ft)

PVC tubes can accommodate both CSL tests and gamma-gamma tests. However, PVC tubes tend to de-bond from the concrete more quickly than steel and it is necessary to perform CSL testing within a few days of casting the shaft. Steel tubes will resist de-bonding for a longer time period (typically at least two weeks), providing more flexibility in terms of when the tests are conducted. However, metal tubes are not considered suitable for gamma-gammma testing without being calibrated for the specific tubes. Regardless of the material, tubes for CSL testing must be filled with water so the acoustic signal can be transmitted from the wall of the tube into the probes and vice versa. Water also helps to maintain temperature equilibrium between the tubes and the concrete, inhibiting the tendency for de-bonding from the concrete due to differential thermal expansion and contraction. A recommended practice is to fill the tubes with water prior to concrete placement to assist in resisting buoyancy forces from the fluid concrete. Filling the tubes with water also maintains temperature equilibrium during concrete curing.

Debonding of the concrete from the smooth-wall CSL access tubes can cause a false indication of an anomaly. Debonding is likely the result of thermal shrinkage or possibly physical displacement at the top of the CSL tubes during concrete curing. Since this issue is typically limited to the top section of the drilled shaft, a visual inspection of the concrete at cut-off level can help determine whether debonding or poor quality concrete led to the anomalous test data.

Initially, the tube pairs to be tested should include at least all perimeter pairs and the main diagonals. If an anomaly is detected, all of the possible pairs of tubes should be tested.

The CSL tubes can also be used for thermal integrity testing and for Gamma-Gamma testing (plastic tubes only), discussed later in this chapter. Access tubes installed for CSL or other NDT methods can also be used for coring the base of a drilled shaft to investigate the quality of the base contact, or for post-grouting at the base if required by the design or as a remediation measure, if necessary. For applications that require

drilling through the bottom of the access tubes, it is advisable to use PVC or other plastic caps at the bottom of the access tubes, which are easy to drill out.

Crosshole Tomography: The Crosshole Tomography (CT) method is based on the same principles as CSL and utilizes the same equipment and access tubes. CT differs from conventional CSL in the way measurements are obtained and in processing of the data. Measurements are made for a larger number of transmitter-receiver locations, including measurements for which the transmitter and receiver are at different elevations (vertical offset). For example, as shown in Figure 16-3, the receiver probe can be positioned at a fixed elevation in one tube, while the transmitter elevation is varied in one of the other tubes. Measurements are made for numerous combinations of vertical offset and for all possible tube combinations. It is typical for measurements in a single shaft to involve tens or hundreds of transmitter-receiver combinations, resulting in the generation of data for thousands of ray paths.

Data generated by CT measurements are analyzed numerically, using a matrix inversion procedure, to generate two- or three-dimensional images of signal velocity, referred to as tomograms. Specialized computer software is required for this type of analysis. A 2-D tomogram is either a vertical or horizontal slice of the drilled shaft between the respective tube pairs showing the measured wave velocities as color contours. Figure 16-4 shows examples of 2-D vertical slices produced by CT testing of a shaft with four access tubes (Hollema and Olson, 2002; Olson, 2005). The tomograms shown correspond to tube combinations 2-3, 3-4, 1-3 and 2-4. Color contours were selected to show sound concrete in green and potentially defective materials in red, orange, and yellow. The perimeter tomograms (2-3 and 3-4) show pronounced zones of low-velocity material, indicating weaker concrete or soil inclusions. A horizontal slice, or 2-D tomogram of the shaft cross-section, is better at showing the lateral extent of an anomaly and can be used to determine where coring might be conducted or to aid in remediation efforts. Tomography results can also be presented in 3-D body diagrams.

CT methods are more time consuming and expensive than CSL testing. A reasonable approach is to conduct CSL testing for routine quality assurance. When CSL results indicate anomalous velocity or energy readings, in which VR values exceed 20 percent, CT methods provide a means to characterize further the size and nature of the anomaly. For example, the CT tomograms shown in Figure 16-4 were developed as a result of CSL tests on a drilled shaft that showed velocity anomalies at depths between 36 and 39 ft from the top of the shaft. The CSL anomalies were observed in the access tube pairs identified above (2-3 and 3-4) and showed velocity reductions in the range VR = 14 to 26 percent (Hollema and Olson, 2002). This type of approach provides owners, engineers, and contractors with information needed to assess the consequences of the potential defects on drilled shaft performance and in making better-informed decisions about further tests, such as coring, and remediation methods.

Another approach found to be effective for CT testing involves using the normal CSL equipment, consisting of one transmitter probe and one receiver probe. The probes are placed in the parallel access tubes in the normal manner and data collected first with the probes at zero offset. A protocol is established so that if an anomaly is detected, offset scans are carried out, first with the transmitter higher than the receiver and then again with the receiver higher than the transmitter. This provides a total of three scans at each depth, per pair of access tubes. In most cases this provides a sufficient number of crossing paths through the center to determine the extent and location of any defect in the center core. Using this protocol saves time and money by avoiding additional follow-up CT testing and the associated mobilization costs.

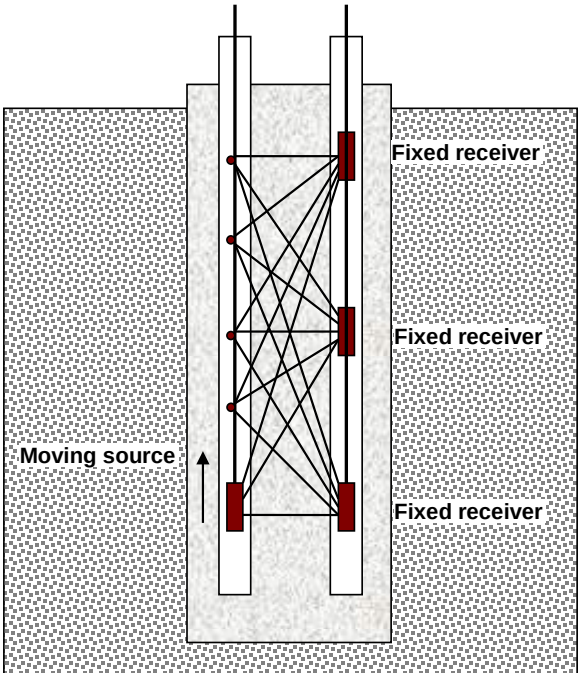


Figure 16-3 Crosshole Tomography Test (after Hollema and Olson, 2002)

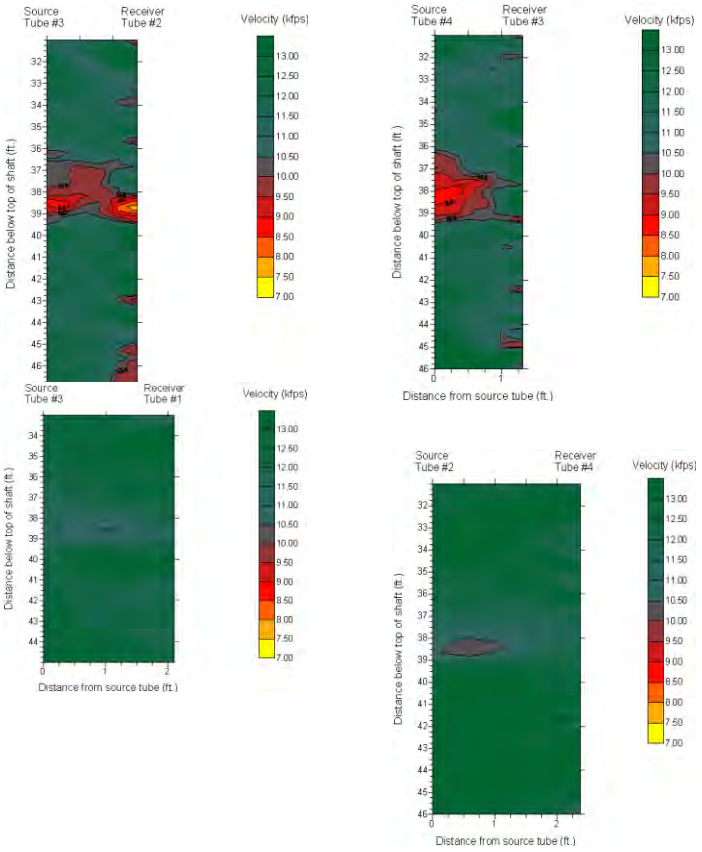


Figure 16-4 2-D Tomograms for a Shaft with Four Access Tubes (Hollema and Olson, 2002)

Perimeter Sonic Logging (PSL): Perimeter sonic logging is the application of cross-hole sonic logging methods with access tubes placed outside the rebar cage. The intent is to evaluate the integrity of concrete around the perimeter of the reinforcing cage. Samtani et al. (2005) describe the results of PSL testing on twenty drilled shafts constructed under polymer slurry in Tucson, where concrete conditions around the perimeter of the shafts were of concern. Numerous anomalies were indicated by the PSL results and test shafts were exposed to confirm the anomalies. The authors report that each suspected anomaly was verified, validating the method as a useful NDE tool. The authors also report that sonic velocities from PSL measurements differ from CSL measurements between tubes located inside the reinforcing cage and exhibit more "noise" than standard CSL measurements. At this time, PSL has not found wide application, but is an option. A practical issue to consider is that tubes on the outside of the rebar cage are difficult to protect during cage placement.

16.2.2 Thermal Method

Thermal Integrity Profiling (TIP) for drilled shafts was introduced by Mullins and his co-workers (see Mullins, 2010; Mullins and Winters, 2011) and is now a widely-accepted method described in ASTM D7949. The temperature of a drilled shaft increases following placement of the concrete, as a result of the exothermic chemical reaction between water and the cementitious components of the mix (primarily Portland cement), referred to as the heat of hydration. By monitoring the temperature distribution within the cross section and over the length of a drilled shaft as the curing process occurs, several integrity characteristics of the concrete can be inferred. These include: locations of potentially defective concrete; geometry of the cross section; and alignment of the reinforcing cage.

Temperature measurements during the concrete curing period can be made using one of two methods: (1) a thermal probe with infrared sensors inserted into access tubes attached to the rebar cage, or (2) embedded cables capable of measuring temperature (thermal cables) attached to the longitudinal bars of the cage. The use of thermal cables is the most practical and widely-used method because it is conducive to automated measurements and does not necessarily require personnel to make measurements manually. Each cable has temperature sensors typically spaced at 12 inches along the cable length. The cables are attached to the longitudinal bars of the cage using zip ties, with typically one cable per foot of drilled shaft diameter, at a uniform circumferential spacing. Figure 16-5 shows a reinforcing cage with thermal cables (yellow) attached to the longitudinal bars. The white boxes labeled TAP are data loggers, and the white box labeled TAG collects the TAP readings via Bluetooth and transmits the readings to the cloud once per hour.

The expected temperature distribution in a cylindrical concrete drilled shaft embedded in the ground is idealized in Figure 16-6. The ground surrounding the shaft provides a thermal boundary across which heat generated in the shaft is transferred to the ground by conduction. As shown in the cross-section view on the left, maximum temperatures occur in the center of the shaft, decreasing radially toward the shaft-soil boundary. Boundary conditions at the top and bottom surfaces of the shaft alter the temperature distribution in the vicinity of these boundaries. Convective and radiation heat loss occurs at the top boundary, causing a vertical profile of decreasing temperature toward the top, as illustrated in the right-hand part of Figure 16-6. Heat loss across the base of the shaft is predominantly conductive, and also generates a vertical profile of decreasing temperature toward the base. The profiles of decreasing temperature at the top and bottom of the shaft are sometimes referred to as temperature 'roll-off.' Groundwater movement, if it occurred, could cause additional convective heat loss along the side or base of a drilled shaft during the curing period. In the portion of a drilled shaft extending through moving water, for example in a river crossing, convective heat transfer could be the dominant mechanism.



Figure 16-5 Thermal Cables Attached to Rebar Cage for TIP Measurements

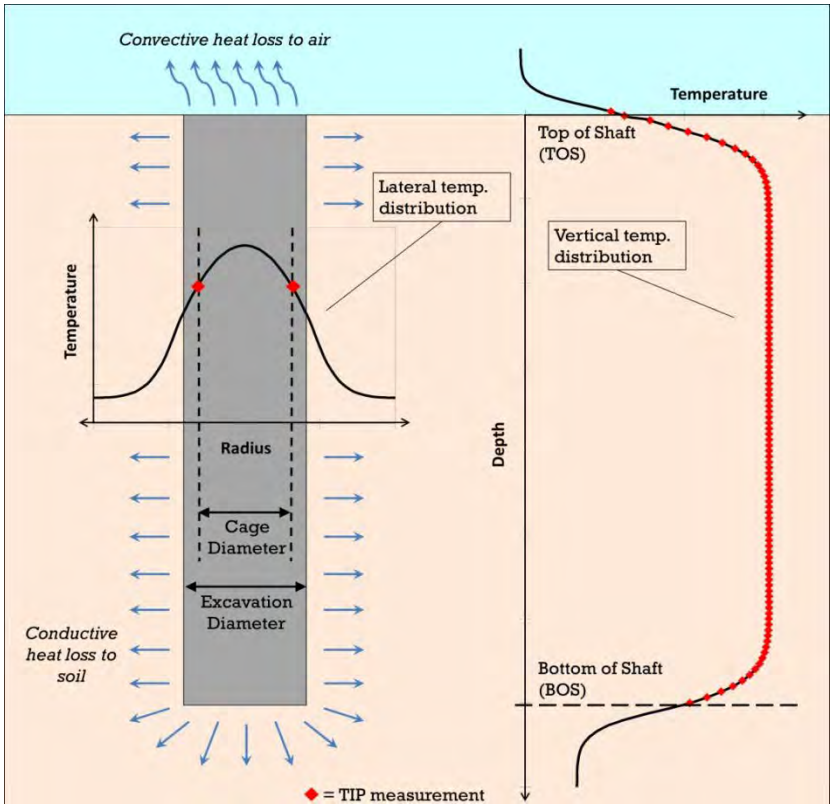


Figure 16-6 Idealized Expected Temperature Distribution in a Drilled Shaft During Concrete Curing (Source: GRL, Inc.)

Inferred properties of drilled shaft concrete are based on interpretation of the measured temperatures compared to the idealized temperature distribution shown in Figure 16-6. TABLE 16-3 summarizes the characteristics determined by TIP and the required measurements. Each feature is then described briefly.

TABLE 16-3 DRILLED SHAFT INTEGRITY FEATURES DETERMINED BY TIP

Concrete/Shaft Integrity Feature	TIP Measurement Used to Infer Feature
Potentially defective concrete	Localized decrease in temperature
Radius and concrete cover at a given elevation	Comparison of local temperature to overall average temperature in combination w/ total volume of concrete placed
Cage alignment	Comparison of temperatures at diametrically opposite thermal measurements

Potential Defects

A plot is generated showing all the temperature measurements from each thermal wire versus depth, as illustrated in Figure 16-7(a). An approximately uniform distribution over the shaft length with only minor and uniform differences between individual cables is indicative of uniform concrete. Temperature decreases, in a single cable or a group of cables over the same depth interval, such as illustrated at a depth of approximately 89 ft in Figure 16-7(a), indicate zones of potentially defective concrete. It is inferred that the lower temperature is a result of lower cement content, which would be expected in zones where concrete mixed with groundwater, soil, drilling fluids, or some combination of these, each of which can result in low-strength or otherwise defective concrete.

Shaft Geometry

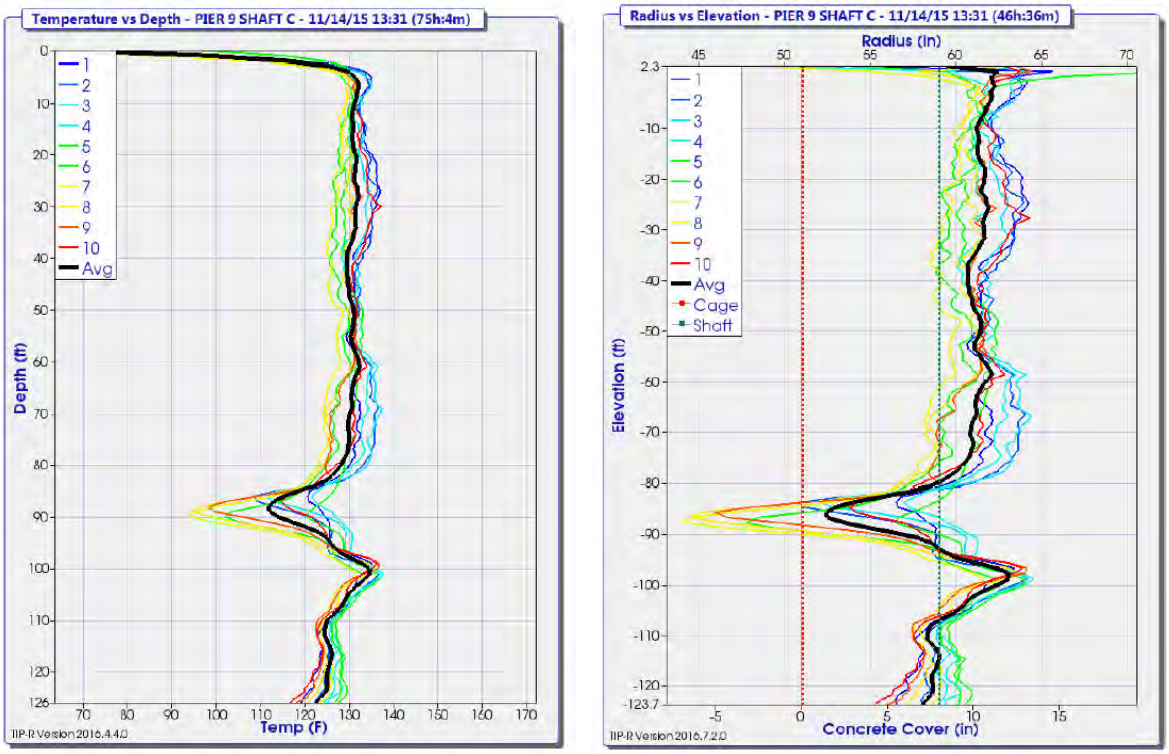
A relationship is first established between the average recorded temperature and average shaft radius. The average radius is determined from the known volume of concrete placed as a function of depth, which requires a concrete volume curve as described in Chapter 15. Based on this average relationship, each temperature measurement is converted to an inferred radius at the depth of the measurement. A plot is generated showing, for each cable, the inferred shaft radius versus depth. Figure 16-7(b) illustrates a radius versus elevation curve. A measured temperature lower than the overall average temperature is interpreted as a local reduction in shaft radius, while an increased temperature relative to the overall average is interpreted as a local increase in shaft radius, *i.e.,* a bulge. Decreases or increases in inferred radius can be used to infer corresponding local decreases or increases in concrete cover of the rebar cage, as plotted on the bottom horizontal axis of Figure 16-7(b).

Rebar Cage Alignment

A comparison between temperature measurements obtained from diametrically opposite locations can be used to infer the position of the cage relative to the shaft perimeter. This is based on the concept that a decreased temperature at one location accompanied by an approximately similar temperature increase at the diametrically opposite point on the other side of the cage represents decreased cover on the lower temperature side and a corresponding increased cover on the opposite side. The logical inference is that the cage is not centered, but rather shifted toward the side of the lower temperature.

Offset of the reinforcing cage would be indicated by a consistent pattern of thermal measurements, or a

gradual change in thermal measurements, over some length of the drilled shaft, reflecting the stiffness of the reinforcement cage. Sharp local changes in the thermal readings would typically not be characteristic of a shift in the reinforcing cage.



(a) Temperature Profile (b) Shaft Radius and Concrete Cover Profile

Figure 16-7 Temperature, Shaft Radius, and Concrete Cover versus Depth Profiles Showing Anomalies at Depth = 89 Ft (Source: Piscsalko and Hannigan, 2018)

Advantages and Limitations

The principal benefit of the TIP method is that it provides verification (quality assurance) of concrete in the entire cross section of the drilled shaft. This overcomes the limitation of CSL, which only measures velocity of concrete inside the rebar cage (core) and gamma-gamma, which is capable of detecting concrete density only in a zone extending several inches radially from the access tube (see next section). A second advantage of TIP is that the test can often be completed and results analyzed within 2 to 3 days following completion of the shaft, providing a shorter time for approval.

Limitations of the TIP method are that the measurements are only possible during the period of elevated temperature, a time window which exists for only several days following the pour. There is no opportunity to perform follow-up testing in cases where there is reason to question the results or if equipment failures occurred. With proper management this is not a major risk, but if the opportunity is missed for any reason, TIP does not provide a second chance. Also, it is not possible to assess the increase in concrete strength or quality over a prolonged time period, or for post-repair testing to assess the effectiveness of shaft remediation measures.

It is possible to have defective concrete at the base of a drilled shaft that is not clearly identified by TIP.

The temperature roll-off at the base, which is anticipated, can make it difficult to discern additional temperature decreases resulting from concrete mixing with cuttings, drilling fluids, etc. that may cause weak or otherwise defective concrete. Since the base of a drilled shaft is a common location of defects for shafts placed in the wet, it is not advisable to rely solely on TIP to provide unambiguous quality assurance on concrete integrity at the base of shafts installed using the wet method. Greater emphasis should be placed on proper base cleanout and its verification, close observation and inspection during tremie placement of concrete, especially the initiation of concrete delivery through the tremie to the base, and other practices recommended in this manual for successful placement in the wet. For drilled shafts constructed using the wet method, consideration should be given to using an alternative NDT method that can better assess conditions at the base of the shaft.

In summary, TIP provides a reliable, practical tool for integrity testing of drilled shafts. TIP overcomes some of the limitations of the other commonly used NDT methods, primarily in its ability to verify the integrity of the entire cross section in contrast to only portions of the cross section tested by CSL and GG. TIP provides additional capabilities including estimated concrete cover and cage alignment. Its limitations, which include the limited window of time for performing the measurements and some ambiguity in detecting anomalies at the base, are manageable and do not detract from its significant advantages. Every method offers some advantages but also has limitations. Used properly within the context of an overall program of quality construction and inspection, TIP is a valuable tool for quality assurance of drilled shafts.

More in-depth integrity evaluation using TIP measurements is also possible by numerical modeling of the temperature profile of the shaft-soil system and signal matching of the model results to the test measurements. Modeling of the temperature distribution requires information on the concrete mix design and the soil profile to predict heat generation and soil insulation parameters. The model output predicts shaft temperatures as a function of time based on an inverse method which is described further by Kranc and Mullins (2007). This type of analysis would be warranted only in special cases where a very detailed investigation beyond the scope of typical project QA is deemed necessary.

16.2.3 Gamma-Gamma Method

The Gamma-Gamma Logging (GGL) method is based on measurements of the rate at which gamma particles (photons) emitted from a source travel through concrete, as measured by a gamma-ray detector. The source and detector are located in a single probe, as shown in Figure 16-8. The probe is lowered to the bottom of a PVC access tube. Gamma rays are emitted from the radioactive source (Cesium-137) while the probe is retrieved at a constant rate. Logging consists of measuring the rate at which photons are detected by a NaI scintillation crystal in the detector. The measured detection rates are referred to as 'gamma ray counts' and are recorded in counts per second or *cps*. Gamma ray counts depend upon the transmission path, the degree of gamma-ray scattering, and gamma-ray absorption. Scattering and absorption depend upon the electron density of the medium surrounding the probe and can be correlated to average mass density of the medium. For concrete, there is a reasonably linear relationship between mass density (or unit weight in lb/ft^3) and the natural log of the gamma ray counts in *cps*.

The radius of influence of a GGL probe is governed approximately by one-half the vertical spacing between source and detector, and the strength of the gamma ray source. For a low energy (10 millicurie) source with a 6 to 8-inch source-detector spacing, the effective radius of penetration is approximately 2 to 3 inches, while a higher energy (100 millicurie) source with a 12 to 18-inch source-detector spacing provides approximately 6 to 8 inches of penetration. Accounting for the diameter of the access tube (typically 2 inches), the gamma/gamma log will provide density information for a relatively small cylinder of concrete concentric with the tube. Probe characteristics vary between manufacturers, and each probe must be calibrated to establish the relationship between gamma ray counts and concrete density (lb/ft^3). For example, Caltrans

Specifications require a GGL probe with a specified radius of detection ranging between 3.0 and 4.5 inches.

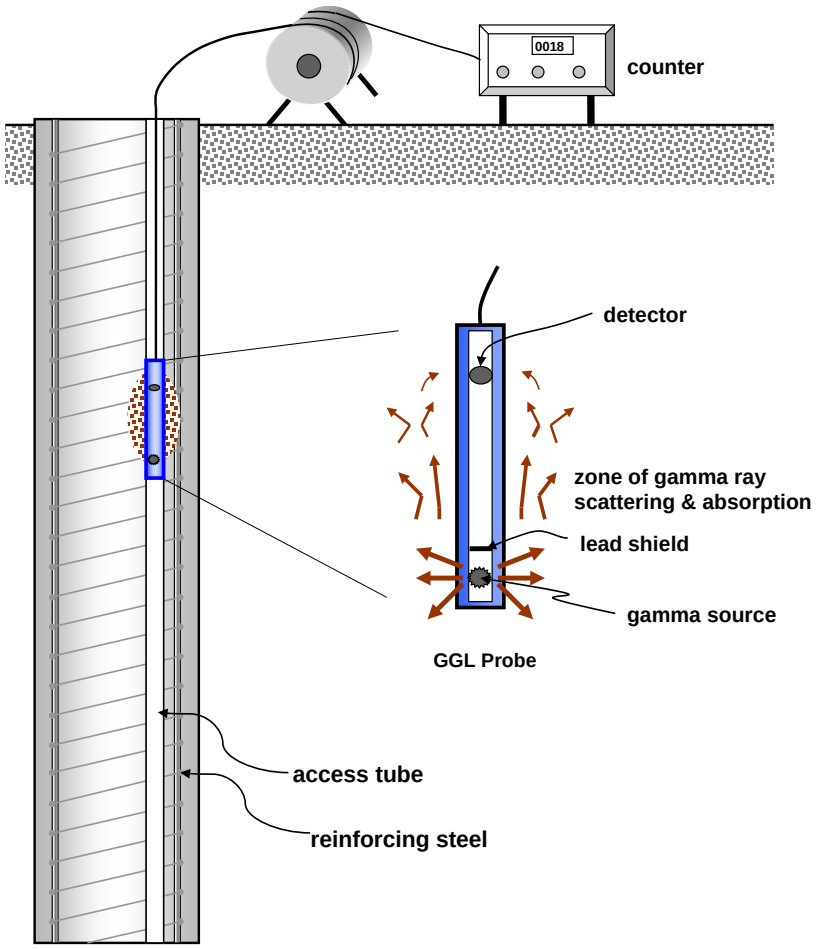
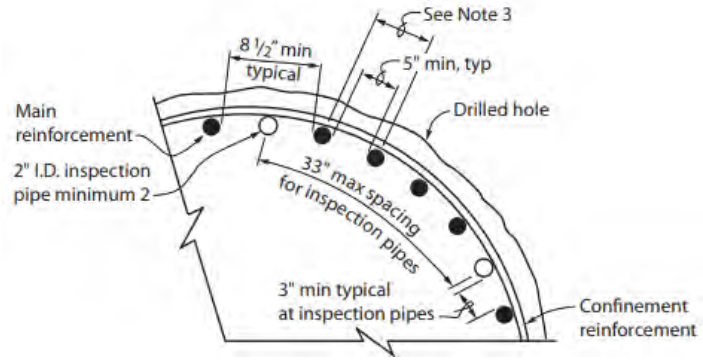


Figure 16-8 Gamma-Gamma Logging (GGL) in a Drilled Shaft

The GGL method for drilled shafts provides a means to evaluate the uniformity of concrete density in a zone adjacent to each tube. Caltrans Specifications require the access tubes to be placed in contact with the inside of the outermost hoop or spiral reinforcement, as shown in Figure 16-9(a). To maintain at least 3 inches of clear space for concrete placement, the adjacent longitudinal bars are specified to have a circumferential spacing of 8½ inches. The specified clear spacing (3 inches) is approximately the radius of influence for the GGL probe, providing a measurement of concrete density in a zone that covers the inside perimeter of the cage, thus providing measurements in the very important zone of concrete coverage. Tubes also must be a sufficient distance from the outside perimeter of the shaft to avoid detecting changes in the density of the soil or rock exterior to the shaft concrete. This requirement also favors the placement of the tubes inside the reinforcement cage. Figure 16-9(b) shows a cage being placed with gamma gamma PVC access tubes attached as specified in the detail shown in part (a).



(a) Caltrans detail for access tube placement



(b) Rebar cage with PVC tubes for GGL

Figure 16-9 Placement of PVC Access Tube In Reinforcing Cage

Evaluation of concrete uniformity is accomplished by defining anomalies in terms of deviations in the measured density. If it is assumed that concrete density follows a normal (Gaussian) distribution, the probability that a density measurement will be three standard deviations below the mean is 1.35 percent. This probability is considered sufficiently low that GGL density measurements less than three standard deviations below the mean are considered anomalous. To illustrate the application of this concept in practice, Figure 16-10 shows typical results from GGL testing on a drilled shaft with four access tubes within a 4-ft diameter cage. The graph shows density on the horizontal axis versus depth on the vertical axis. The mean density from all data points within the shaft is computed and shown on the graph as a vertical trend line. The standard deviation (σ) is calculated and a line corresponding to 3σ below the mean is also shown on the graph. Any location where consecutive readings over a 6-inch interval in any of the tubes are below the mean minus three standard deviations (mean -3σ) is defined as anomalous. The mean and standard deviation are based on all of the readings taken in the shaft, excluding anomalous intervals. There is currently no ASTM standard method for GGL testing; however, Caltrans publishes a test procedure designated as 'California Test 233' (Caltrans, 2005).

The shaft of Figure 16-10 was installed under slurry. The gamma-gamma log shows anomalous readings near the base of the shaft, possibly because of mixing of slurry or sediment with the concrete. For this particular shaft, the possible contamination of concrete near the base was not judged to be severe enough to warrant consideration. However, there is a zone higher up the shaft, where a marked reduction in concrete density occurs in two of the four tubes. Additional investigation showed defective concrete at this depth.

Several features of drilled shaft reinforcing cages and access tubes may affect GGL density measurements, but are not related to concrete density. For example, the distance between the access tubes and the steel reinforcing must be uniform along the length of the tubes so the effect of steel on the measured density is consistent. Readings will also be affected by couplers at the PVC tube joints, which may appear as minor anomalies in density. For proper interpretation, the contractor or testing agency should provide a log

indicating the elevation of couplers. Since couplers typically are at 20-ft intervals, their locations should be easy to recognize. Other features that may affect GGL readings include changes in the reinforcement schedule, the presence of instrumentation or load cells, and ties or any other fixtures attached to the reinforcement. In soft soils, the plastic wheels used to center the cage could penetrate the sidewall soil, which may show up as an anomalous reading. It is important to document the location of all such features.

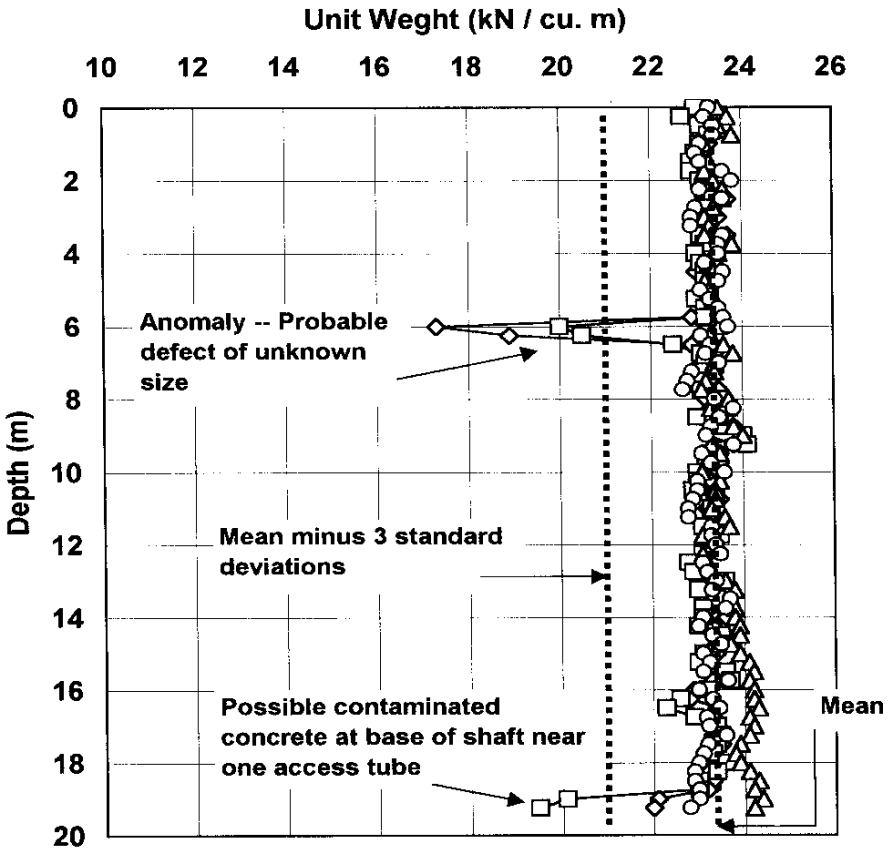


Figure 16-10 Results from Gamma-Gamma Logging of a Drilled Shaft with Four Access Tubes (Courtesy of Caltrans); 1 m = 3.281 ft., 1 kN = 224.8 lb.

GGL readings are reliable and repeatable, and they are not affected by jobsite ambient vibrations, electrical interference, or by debonding of the access tubes. Data processing is relatively straightforward and there are no limitations on depth. The method is effective in identifying anomalies that occur within several inches of the access tubes. The primary application of GGL is in detecting anomalies and defects in the concrete coverage zone for the steel reinforcing cage. Anomalies in this zone are often the result of soil caving or poor concrete flow through the rebar cage. These can be critical defects because they expose the steel reinforcement to corrosion. According to Caltrans (2010), the time required to conduct GGL testing depends on the size of the drilled shaft. Each inspection tube is logged once, and a drilled shaft roughly has one tube per foot of shaft diameter. During logging, a GGL probe is raised at a rate between 10 and 15 ft per minute. A typical 6-ft diameter, 100-ft long shaft requires approximately 2 hours to conduct logging, excluding analysis time. An example of the type of defect which GGL is highly suitable for identifying is shown in Figure 16-11. Low-density concrete or lack of concrete cover of the rebar as seen in this photo will be detected by GGL.



Figure 16-11 Example of Concrete Cover Problems Readily Detected by Gamma Gamma Logging

GGL testing has several limitations. The zone of concrete that is tested is a relatively small portion of the shaft cross section and does not extend to the interior of the shaft. For example, approximately 7 percent of the cross-sectional area of concrete is tested for a 6-ft diameter drilled shaft. When the tubes are placed inside the reinforcing cage, which is the most common location, GGL may not detect anomalies or defects in areas outside of the cage if they do not penetrate to the outside perimeter of the reinforcement cage. The method is not effective for identifying the lateral extent of an anomaly, a capability that is better suited to CSL testing, including tomographic methods. Proper alignment of the PVC tubes requires care during construction to ensure tubes are straight and undamaged. GGL is not effective over a distance of approximately 1 ft above the bottom of each access tube as a result of the bottom boundary condition where the concrete ends. A blocked tube precludes GGL measurement below the elevation of blockage.

The GGL probe contains radioactive material which requires licensing and compliance with Nuclear Regulatory Commission (NRC) regulations, including those applicable to transportation of the equipment. Personnel who handle and operate the GGL probe must be certified and monitored for exposure. Radioactive decay of the source material requires that each probe be recalibrated regularly to account for reductions in the signal strength. These are management issues but should be considered by agencies interested in using GGL for drilled shaft integrity testing.

16.2.4 Methods Based on Analysis of Stress Waves

These tests have not found wide application as quality assurance tools in the transportation industry. However, they have been used on occasion when inspection records raise questions about concrete integrity and the drilled shaft was not equipped with access tubes or other means to use CSL, GG, or TIP for further assessment. Tests included in this group are the Sonic Echo (SE) and Impulse Response (IR) methods. Variations of the SE test include the Bending Wave and Parallel Seismic tests. These methods involve the generation of low-amplitude stress waves at the top of the shaft. Properties of the shaft concrete are inferred from measured reflections and travel times of the stress waves. Both the SE and IR tests require an impulse hammer and a motion sensor (either a geophone or an accelerometer) positioned at the head of the shaft. An operator strikes the top of the shaft with the hammer, generating a compressional stress wave that travels

down the shaft at velocities in the sonic range. Wave energy is reflected at any location at which there is a change in impedance, for example at the base of the shaft. The reflected wave travels back up the shaft where it is detected by the motion sensor. Impedance, or resistance to wave propagation, is a function of the shaft cross-sectional area, wave propagation velocity, and mass density of the concrete. Defects or irregularities in a drilled shaft, such as soil intrusions, changes in concrete density, or any change in the shaft dimensions, will change the impedance and result in early reflection of wave energy. Identification and interpretation of wave reflection signals (echoes) form the basis of these methods. The difference between SE and IR methods pertains primarily to the techniques used to process the test measurements. Test results are analyzed in the time domain for the SE method and in the frequency domain for the IR method.

Sonic Echo (SE) Method

The SE method is illustrated in Figure 16-12. Waves reflected from irregularities and/or the base of the drilled shaft are detected by the accelerometer (receiver). A signal analyzer is used to processes and display the hammer and receiver signals as a function of time. The figure illustrates an ideal case where a single reflection from the base is recorded. Digital filtering is usually applied to eliminate high frequency noise and unwanted reflections from the sides of the shaft. Changes in impedance are inferred by identifying and analyzing the arrival times, direction, and amplitude of the reflected signals. For example, for the case illustrated in Figure 16-12, the as-built length of the drilled shaft is found from the simple equivalence shown in the figure and knowledge of the velocity of the compressional wave in concrete, V_c.

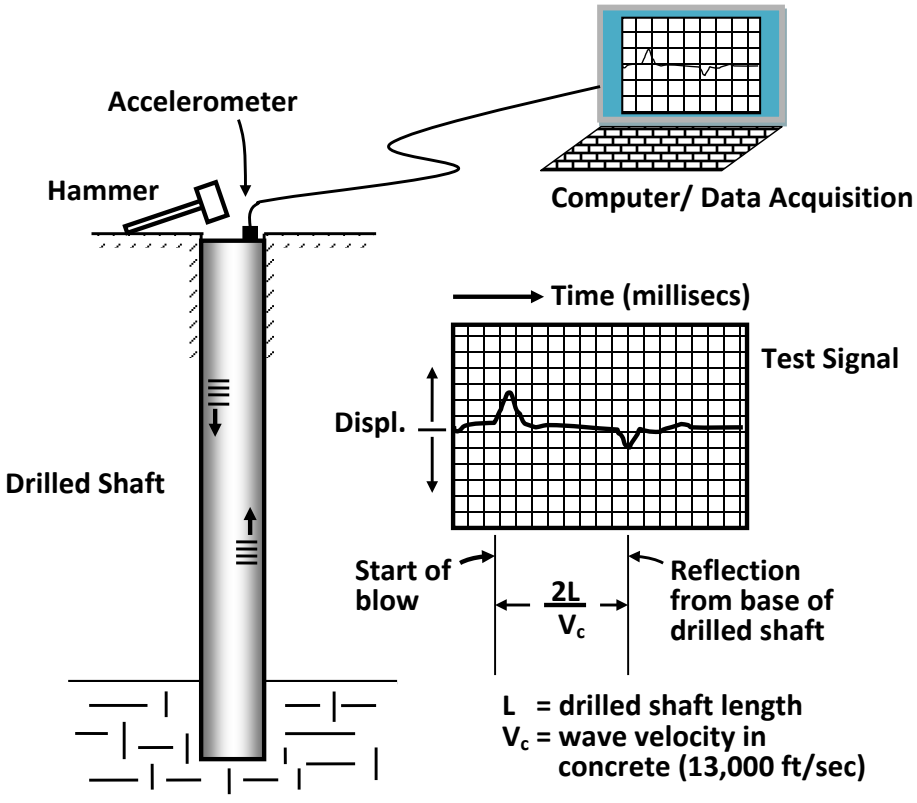


Figure 16-12 Sonic Echo Method

Evaluation of drilled shaft integrity from SE tests requires interpretation of the reflected wave signals based

on the following principles:

- Potential causes of an impedance decrease include soil intrusions, honeycomb or otherwise low density concrete, cold joints or breaks in the shaft, and decreases in the cross-sectional area of concrete;
- A reflection that indicates an impedance decrease is sometimes referred to as a "neck," regardless of its underlying cause;
- Impedance increases may occur due to an increase in shaft diameter or increases in the competency of the surrounding soil or rock, and are often referred to as a "bulb." In general, a bulb is not interpreted as a potential defect.

Identification of a neck does not, by itself, provide definitive proof of a defect in a drilled shaft, nor does it provide a means to determine exactly which property of the shaft has caused the decrease in impedance. For this reason, reflections other than those caused by the base of the shaft should be identified as anomalies. It is then up to the engineer to decide whether a detected anomaly represents a potential defect. Independent observations made during construction provide the additional evidence needed to classify an anomaly as a potential defect. For example, if the anomalous reading (neck) corresponds to a depth at which caving was observed or a concrete underpour occurred (based on concrete volume monitoring), this information in combination with the anomalous SE signal would suggest a potential defect. In addition, the interpreter of the SE test must be familiar with its limitations, as follows:

- The strength of the echo depends on the surrounding soil or rock. For this reason the interpreter of a sonic echo or similar test (impulse-response, impedance logs) must have access to the boring data and be familiar with the subsurface conditions at the site of the test shaft.
- Echoes are frequently too weak to be distinguished when drilled shaft length to diameter ratios exceed approximately 10:1 in rock, 20:1 in stiff or hard soils, 40:1 in medium-stiff soils, or 60:1 in very soft soils.
- The reflection (echo) from the base of a drilled shaft in rock may be weak, especially if there is a clean contact between the concrete and rock.
- The smallest size of detectable anomalies is approximately 10 percent of the drilled shaft cross-section (Iskander et al., 2001).
- The geometry of a defect (percentage of the cross section, thickness, and position within the cross section) generally cannot be determined. However, in larger-diameter shafts, the test can be conducted in several locations around the top of the shaft, making it possible to identify and locate reflections occurring in limited portions of the cross-section over a depth of several diameters (3 to 5) from the top of the shaft. Often this is a critical zone for defects, especially for shafts under lateral loading.
- Defects located below the uppermost major defect exhibit weaker reflections (shadow effect) and the ability to identify multiple defects is limited; a major defect will make it impossible to detect anything below it.
- Defects at or near the shaft base cannot generally be identified because of the uncertainty in the material wave speed and the inability to distinguish between a reflection from a normal base and a reflection caused by a defect in the same location.
- Planned or unplanned diameter changes can appear as anomalies even if the diameter is acceptable; for example, a reflection will appear at the change in diameter between the cased portion of a shaft above rock and the uncased portion below top of rock.
- Impedance changes in response to a change in concrete modulus or density can be caused by slurry contamination or honeycombing in the concrete. However, a similar effect can also result from changing ready mix trucks during a concrete pour in which the modulus and/or density change.

Although detectable, this type of anomaly does not constitute a structural defect.

It may be possible in some cases to simulate the size, location, and nature of a defect using a one-dimensional wave equation program. The size, position, and stiffness of the defect are varied in the computer code in order to match the computed velocity time history at the head of the shaft with that measured by the test. As described by Middendorp and Verbeek (2005), this simulation sometimes allows for a better understanding of the possible properties of the defect. However, the "curve matching" procedures are not unique.

In summary, the sonic echo test should be considered as a screening method that is capable of locating defects covering at least 10 percent of the cross section, such as voids or soil inclusions, and bases of shafts that were drilled to the wrong depth. The method is most useful for investigating potential defects in shafts that exhibit unexpected post-construction performance problems (*e.g.*, excess settlement) but which are not instrumented with access tubes for CSL or GGL methods. SE can also be used to confirm defects identified by CSL or GGL testing.

Impulse-Response (IR) Test: The IR method is similar to the sonic echo test in that a stress wave is generated by hammer impact at the head of the shaft. The hammer is equipped with a built-in load cell that can measure the force and duration of the impact. The method of processing the test measurements is described, among others, by Finno and Gassman (1998). Both the force and measured velocity readings are transformed to the frequency domain by performing a fast Fourier transform (FFT). The resulting velocity spectrum (V) is divided by the force spectrum (F). This ratio (V/F) is referred to as the "mobility" and is plotted on the vertical axis against vibration frequency, in Hz, on the horizontal axis. The resulting graph, called a mobility plot, is used to evaluate and interpret reflections caused by changes in shaft impedance. Figure 16-13 shows an idealized mobility plot. The following parameters are defined from the plot: P and Q = mobilities corresponding to the local maximum and minimum values of the resonant peaks, respectively. The drilled shaft mobility N is defined as the geometric mean of the height of the resonant peaks in the portion of the mobility curve where the shaft is in resonance. As shown in Figure 16-13, the shaft mobility N is a function of P and Q, and can also be defined theoretically as the inverse of drilled shaft impedence, where impedence is the product of density (ρ_c), compression wave velocity in concrete (V_c), and cross-sectional area of the drilled shaft (A_c).

The principles applied to the interpretation of mobility plots are as follows. First, the initial slope of the curve is related to the low-strain axial stiffness of the drilled shaft. If the low-strain stiffness is low compared to those of other shafts of the same size and in the same ground conditions that are known to be sound, the reading is anomalous. Second, reflections appear as resonant frequency peaks on the mobility plot. The distance between frequency peaks (Δf) is controlled by reflections at locations where there is a change in impedance. In Figure 16-13, the reflections are from the base of the shaft, in which case the mobility plot provides a means to determine the as-built length of the shaft using the relationship shown in the figure between frequency change Δf, wave velocity in concrete V_c, and drilled shaft length L. If reflections occur at locations other than the base, the relationship shown in the figure can be used to estimate the distance Z from the geophone to the source of the reflection by Equation 16-5:

$$Z = \frac{V_c}{2\Delta f} \qquad 16\text{-}5$$

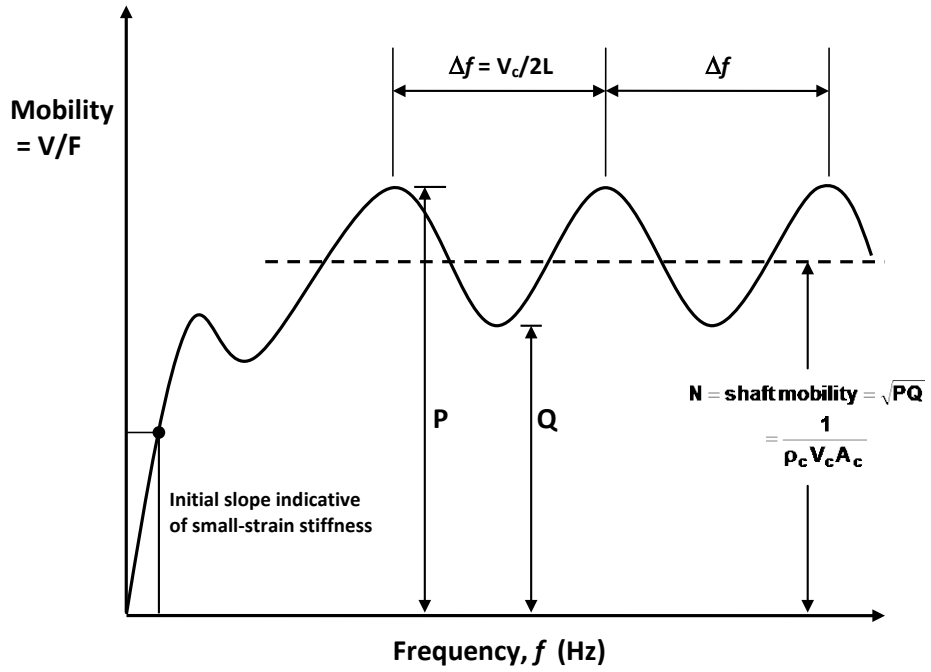


Figure 16-13 Ideal Mobility Plot from Impulse Response Test

Finally, the shaft mobility, *N* (dashed line), can be related to the average cross-sectional area of the shaft if values of modulus and density are assumed. If the measured value of *N* from the mobility plot is greater than the calculated value, the reading is considered anomalous and a defect could be present. The anomaly could be due to a decreased cross-sectional area (A_c) or poor concrete quality causing a decreased density (ρ_c) or velocity (V_c), or both.

Finno and Gassman (1998) identify some of the factors that can limit the applicability of the IR method for drilled shafts. The capability to locate the base of a drilled shaft is limited by the resolution of the mobility plot. Resolution is defined in terms of the ratio *P*/*Q*. When the *P*/*Q* ratio approaches 1.0 (the minimum and maximum peaks shown in Figure 16-13 are very close), no resonant frequency peaks can be distinguished, and it is therefore not possible to identify reflections whether they come from the base of the shaft or other locations. The *P*/*Q* ratio that can be achieved is shown to depend on the length to diameter ratio of the drilled shaft (*L/B*), the ratio of soil shear wave velocity to propagation velocity of the concrete, and the ratio of soil density to density of the concrete. When these values can be estimated or measured, the user can assess whether the IR method will provide meaningful results for a given drilled shaft and surrounding soil conditions. Further details are given by Finno and Gassman (1998). Under most circumstances, the base of a drilled shaft cannot be identified for *L/B* greater than 30.

Certain construction details can affect the mobility plot obtained from IR testing and must be known by the test interpreter to be taken into account properly. This includes features such as the bottom of permanent casing, the presence of loose fill in the annulus between temporary and permanent casing, overexcavation of the drilled shaft, and soil disturbance caused by drilling (Davis and Hertlein, 1991; Finno and Gassman, 1998; Hertlein, 2009).

Impedance Log: An impedance log is a two-dimensional representation of a drilled shaft based on signals obtained from IR testing. The measured response is compared to the simulated response of an ideal infinitely long drilled shaft in soil. The simulated response is adjusted and scaled based on this comparison, then integrated to produce an impedence log. Details of the numerical analysis are beyond the scope of this

manual but the interested reader is referred to Hertlein and Davis (2006) or Gassman and Li (2009). Impedance decreases are represented as equivalent decreases in shaft diameter (a neck), while impedance increases are represented as equivalent increases in shaft diameter (a bulb). Examples of impedance logs are shown in Figure 16-14 (Gassman and Li, 2009). Parts (b) and (c) of the figure show best-estimate and best-fit impedence logs, respectively, for a drilled shaft constructed with a soil-filled joint over the depth interval shown in part (a). Several informative case histories are presented by Hertlein (2009) describing the application of IR testing and impedence logs to evaluation of drilled shafts.

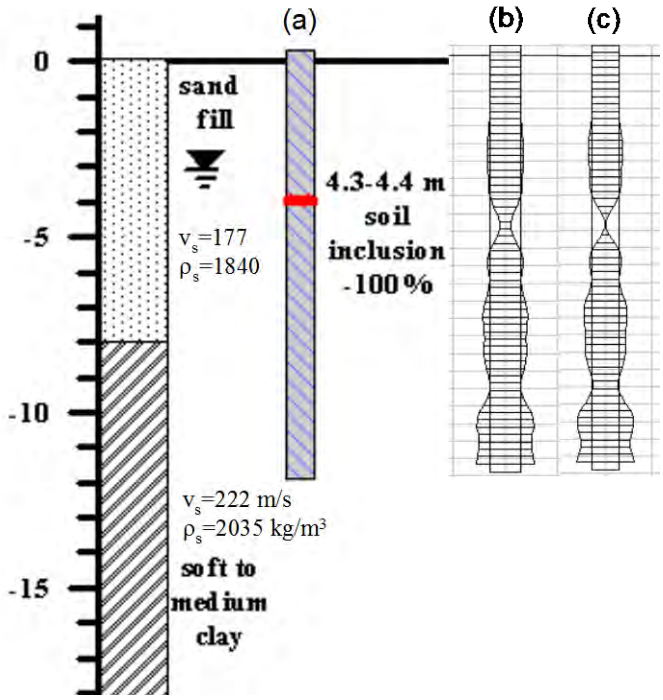


Figure 16-14 Examples of Impedance Logs, Drilled Shaft with Known Inclusion (Gassman and Li, 2009); 1 ft = 0.3 m; 1 lb/ft^3 = 16 kg/m^3

16.3 DRILLING AND CORING

A frequent response to concern about the integrity of a particular drilled shaft, for example as a result of a problem observed during placement of the concrete and identified by the inspector, or as a result of significant anomalies noted from non-destructive tests, is to execute a program of drilling and/or coring. Core sampling provides direct visual examination of concrete and the opportunity to conduct strength tests on as-placed concrete. However, drilling and coring are time-consuming and expensive. Drilling and coring also have limitations that may preclude the visual verification that is needed or desired in every case.

In most cases it is not necessary to core the entire length of the shaft, and it may be more economical to blind drill competent sections of the shaft since drilling is much faster than coring. An effective way to employ a coring program is to limit core sampling to target zones in the shaft where concrete quality is questionable. In zones that are not cored, the quality of concrete can sometimes be inferred from the drilling rate. Drilling may also reveal defects, for example, a soil-filled cavity may be indicated by a drop in the drill for a significant distance.

Coring in the target zone can provide both qualitative and quantitative information on the integrity and quality of drilled shaft concrete. Visual observation of core samples is useful in identifying voids, weak cementation, fractures, soil or slurry intrusions, and other flaws or defects. Reduced core sample recovery may also indicate defective concrete. Intact core samples can be tested for compressive strength. For coring to be effective, high quality cores must be retrieved. It is recommended that cores be recovered utilizing double or triple barrel techniques. Ideally, core diameters for strength tests should be a minimum of four to five times the maximum aggregate size. For mix designs with 1/2 inch maximum aggregate, NX core is sufficient, but for concrete with maximum aggregate size in the range of 1/4 inch to 1 inch, a 4-inch diameter core is preferred for strength testing.

Figure 16-15 illustrates features observed in concrete core that serve the purpose of either verifying the concrete quality at the cored location, or that reveal flaws or defects. Part (a) shows concrete core in a shaft that exhibited CSL anomalies with velocity reductions in the 15 to 20 percent range. Coring showed continuous, integral concrete over the entire cored depth. Compression tests on core sample exhibited strengths well in excess of the 5 ksi design value. No further action was taken. Part (b) shows core at the base of a shaft obtained in response to questions raised about base cleanout effectiveness; the core at this location showed high quality concrete in direct contact with the supporting rock and no further measures were deemed necessary. Part (c) shows a core sample take from near the top of a drilled shaft that exhibited an excessive amount of bleed following completion of the pour. This sample shows poor cementation and the presence of bleed channels. The solution was to chip down the top of the shaft until acceptable concrete was verified, and re-pour the top portion of the shaft. Part (d) of the figure shows core near the tip of a shaft where difficulties with the tremie pipe resulted in the initial discharge of concrete several feet above the base under wet conditions. Poor recovery and weak concrete observed in the core resulted in remediation by hydrodemolition followed by grouting.

Jones and Wu (2005) note that core drilling can be challenging due to access difficulties and in ensuring the target zone is actually drilled. Core drilling typically is performed using small-diameter rotary equipment that is truck or ATV mounted, although portable equipment is sometimes used when drilling is limited to the top few feet of the shaft. Often, more space is required to set up a truck-mounted geotechnical rig compared to the rig used for drilled shaft excavation. This is because the crew must work behind the rig and have at least 10 ft of clear space behind the core hole. Reinforcing steel extending above the shaft creates a difficulty since most rigs cannot clear more than 3 ft vertically. In those cases it may be necessary to raise the ground by placement of fill or to use crane mats or platforms to elevate the rig.

The ability of a concrete core hole to penetrate precisely the target zone of questionable concrete can be limited by the reinforcing steel, NDT access tubes, and the potential for horizontal drift of the core hole. For example, if the target zone is identified by a CSL anomaly which is limited to the zone between two access tubes or by anomalous GGL readings in the vicinity of a single tube, coring may have limited ability to target these zones. Jones and Wu (2005) indicate that core holes should be located a minimum of 6 inches away from steel reinforcing and steel access tubes to reduce the risk of interference while drilling. At least several inches of horizontal drift can be expected in most core holes (and in the drilled shaft as well), and it is not uncommon for the core hole to run out the side of the drilled shaft or to encounter one or more bars of the reinforcing steel, particularly for deep core holes. Experienced personnel are required, along with appropriate equipment, to have greater confidence that the drilling is done correctly in the direction intended.



(a) Continuous High Quality Core



(b) High Quality Core Showing Base Contact with Rock



(c) Core Exhibiting Poor Cementation



(d) Defective Zone of Weak Concrete and Poor Recovery

Figure 16-15 Example of Features Observed in Concrete Core

Given the above limitations, drilling and coring are most effective for identifying and characterizing defects of relatively large size. If the excavation has collapsed during the concrete placement and if the concrete is absent in a section of the shaft, the defect is almost always sure to be detected. Smaller defects can easily be missed. The reverse can also be true; that is, coring may reveal a defect that is thought to be severe but in fact is insignificant. For example, coring can reveal weak concrete or sand locally at the base of a rock socket, but sound rock and a good contact could exist across most of the socket.

Core or drill holes should be filled with grout or concrete upon completion of sampling if the shaft will be used in a structure.

16.3.1 Downhole Video Inspection

Drilling or coring into drilled shafts, either through access tubes or directly into the concrete, provides the opportunity to employ cameras that can be lowered downhole while providing a video image on a computer or television screen at the surface. A variety of camera types are available for this purpose. The camera shown in Figure 16-16a fits into a 2-inch diameter PVC inspection tube or a core hole and provides real time true color pictures to the surface. Figure 16-16(b) demonstrates the visual information obtained from a downhole video inspection. The photo is from a project where it was used to investigate a zone where coring showed zero recovery, raising questions about the concrete integrity. However the camera showed continuous concrete in the walls of the corehole with good distribution of aggregate, indicating that poor recovery may have been caused by the coring equipment or method, and not a result of defective concrete.



(a) (b)

Figure 16-16 (a) Downhole Camera Used by Caltrans (photo courtesy of B. Liebich, Caltrans); (b) Example of Image Obtained from Corehole Video

16.4 LOAD TESTING FOR DRILLED SHAFT INTEGITY EVALUATION

A field load test on a drilled shaft provides valuable information on some, but not all, aspects of integrity. Successful performance during a load test may verify the geotechnical and structural resistances of a drilled shaft, but only for the specific test conditions and over the relatively short time period of the test. In this sense, any of the load test methods described in Chapter 13 can provide some measure of drilled shaft integrity. A rapid load test, as discussed in Chapter 13, provides a means for measuring drilled shaft axial or lateral response without prior installation of instrumentation (*i.e.*, an unplanned test). High-strain dynamic tests, carried out by dropping a heavy weight onto the head of the shaft from various heights, can also be used to evaluate characteristics of stress wave propagation. Drop-weight load testing interpretation relies on analysis methods similar to those used in dynamic pile testing. Strain gauge and accelerometer measurements are made at the top of the shaft. If sufficient shaft resistance is mobilized, it is possible in theory to relate the stress wave characteristics to drilled shaft strength and stiffness utilizing available dynamic testing technologies.

A full-scale static axial or lateral load test to prove drilled shaft resistance may be warranted when a systematic error has been made in the construction of the drilled shafts for a project. This approach can be effective when a definitive test on one shaft would either confirm the acceptance of all of the shafts or show that the systematic error has affected the performance of all of the shafts.

Not all potential defects will be detectable by load testing. For example, a lack of sufficient concrete cover over steel reinforcing may not influence drilled shaft performance during a load test, but could affect the long-term performance of a drilled shaft if the reinforcement undergoes corrosion.

16.5 SUMMARY

Tests conducted on completed drilled shafts provide a means to verify the integrity of the concrete. The most commonly used tests for this purpose are nondestructive tests (NDT) that are based on measuring some physical characteristic of the hardened concrete. Each test has advantages and limitations, and the results should always be considered in conjunction with construction observations.

This chapter presents an overview of the most commonly used NDT methods for drilled shafts. Cross-hole sonic logging (CSL) is currently the most widely used method. Tomography, based on the same concept and involving the same equipment as CSL, provides enhanced evaluation of potential defects. Thermal Integrity Profiling (TIP) is being used increasingly and is the only method that provides assessment of the full cross section. Gamma-gamma testing is used extensively by several state agencies and is most effective for verifying integrity around the perimeter of a drilled shaft, where concrete cover of the reinforcement is a critical integrity issue. These and other methods are described. Methods to further investigate the integrity of drilled shafts when there is reason to suspect a defect, including coring and load testing, are discussed.

CHAPTER 17
ASSESSMENT AND ACCEPTANCE OF DRILLED SHAFTS

17.1 INTRODUCTION

The final step of a drilled shaft project is official acceptance of the as-built shafts by the engineer in responsible charge. Acceptance of each drilled shaft installation requires official records fully documenting the installation of the shaft, and demonstrating that the as-built shaft conforms to the Plans and Specifications, including those pertaining to post-construction integrity testing. In the event that any of the installation criteria have not been satisfied, additional measures could be required prior to final acceptance. This chapter outlines the process of assessment and acceptance, and provides guidance on the types of information used to evaluate completed drilled shafts. In cases where construction records and/or nondestructive testing (NDT) suggest the possibility of defects, guidance is provided on the follow-up actions needed to further evaluate the nature and extent of potential defects and to assess their effect, if any, on the ability of the as-built drilled shaft to meet the design requirements for strength and serviceability. In cases where it is determined that a drilled shaft is not adequate, methods for remediation and repair are presented.

It is important for all parties involved in drilled shaft design and construction to understand that perfection in drilled shaft construction is an unattainable ideal. Some small imperfections are to be expected and a robust design should be tolerant of small and unavoidable flaws which may or may not be detected. An imperfection in the completed shaft is a deficiency requiring remediation when the shaft is insufficient or inadequate to meet the strength and serviceability requirements during the design life of the structure. To facilitate the consideration of these issues, the following definitions of terms are provided:

- Anomaly - deviation from the norm; an irregularity. This term is often used in reference to an anomalous pattern in the integrity test measurements. An anomaly may or may not represent a defect.
- Defect (noun) - a fault or flaw
- Deficient - insufficient or inadequate
- Mitigation - to make less severe
- Remediation - act or process of correcting a fault or deficiency
- Repair - restore to condition necessary to meet performance requirements

In general, the terminology used in this chapter refers to remediation or repair of a deficiency because the objective is to restore a foundation which is inadequate to one which meets the strength and serviceability requirements of the structure. Mitigation of a defect (making it less severe) might be a sufficient remediation strategy in some cases. Even if an anomaly is detected and determined to represent a defect in the shaft, no remediation might be required if the flaw is determined to be in a location or of extent that the foundation is not deficient.

17.2 ASSESSMENT OF COMPLETED DRILLED SHAFTS

Figure 17-1 outlines the steps that lead to final acceptance of drilled shafts. These steps are an expanded version of Step 21 in the overall process introduced in Chapter 8, titled 'Observe and Evaluate Construction of Production Shafts,' and Step 22, titled 'Post-Construction Evaluation and Report." The information used as the basis of acceptance is divided into two categories: (1) field observations during construction (inspection records), and (2) evaluations of concrete integrity through the use of one or more of the NDT methods described in Chapter 16.

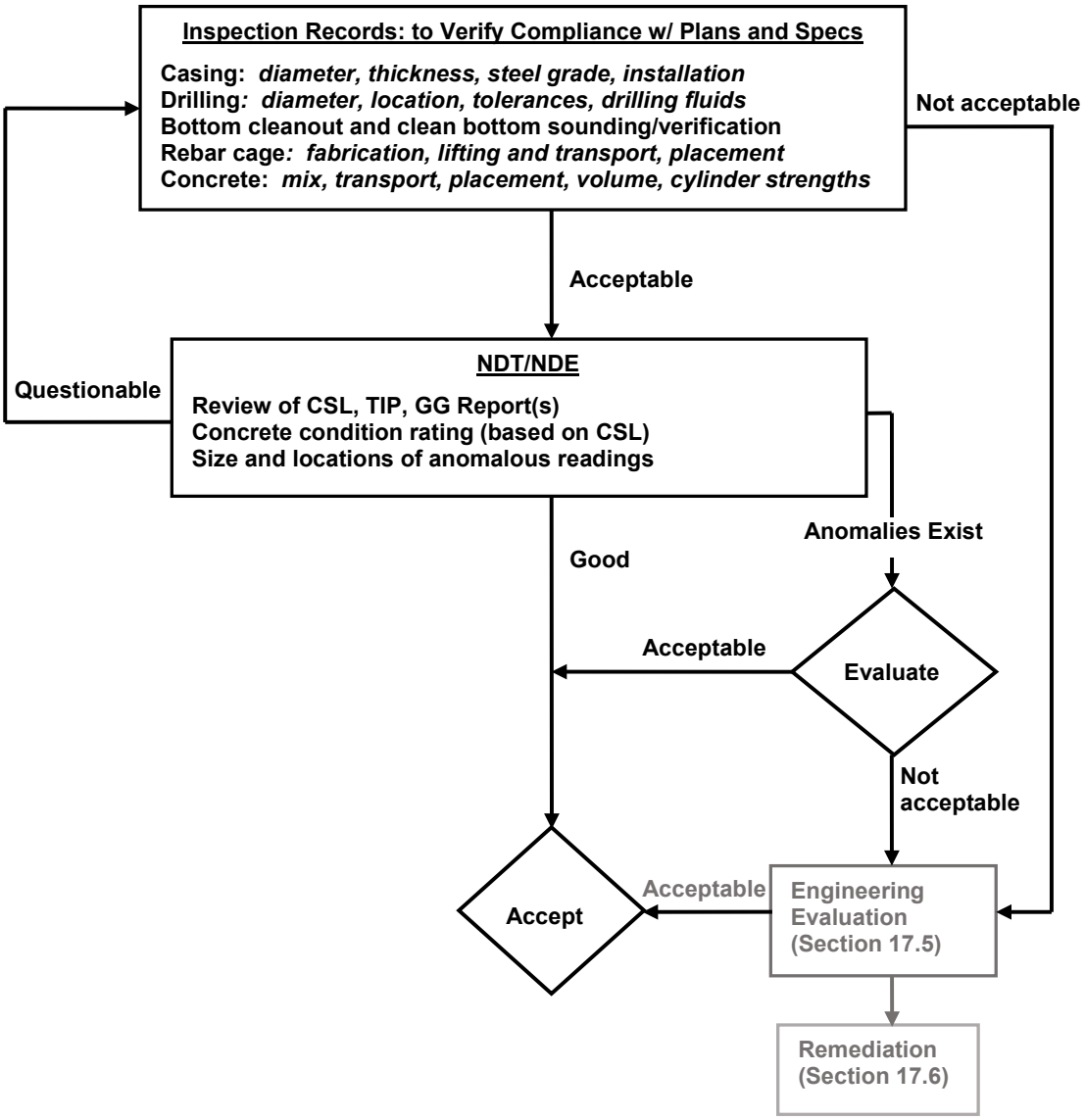


Figure 17-1 Drilled Shaft Assessment and Acceptance Process

17.2.1 Evaluation Based on Installation and Inspection Records

Chapter 15 describes the inspection and documentation of drilled shaft construction. It is emphasized in Chapter 15 that complete written documentation of the work is essential for providing a permanent record of the as-built shafts and is the primary basis of quality assurance for each drilled shaft on the project. The specific forms and details of the information recorded on the forms can vary between agencies and projects, but as a minimum should include records documenting casing inspection and installation, logs of the excavation, properties and handling of drilling fluids, base cleanout, reinforcing cage inspection and installation, concrete testing and placement, and post-construction integrity testing. More detailed guidance on specific information to be documented as part of the inspection is presented in Chapter 15 and also in the Guide Specification discussed in Chapter 14 and presented in Appendix D. Sample inspection forms are included in Appendix E. The most important consideration is that the information

provided in the completed forms is sufficient to confirm the as-built location, dimensions, and elevations of each drilled shaft and that the installation process is sufficiently documented to verify compliance with the drilled shaft construction specifications and the approved Drilled Shaft Installation Plan.

The engineer in responsible charge of the drilled shafts, typically the foundation design engineer, is responsible for reviewing the inspection records and providing a decision in a timely manner. When subsequent construction of columns, piers or other structural elements is contingent upon approval of the drilled shafts, it becomes critical for the inspection records to be prepared as an organized package, provided to the responsible engineer as soon as possible, and then reviewed and either approved or rejected to avoid delays to the schedule.

When the inspection records have been reviewed and it is determined that the drilled shaft meets all applicable criteria, the next step is evaluation of the NDT results. In cases where review of the inspection records and NDT data results in rejection of the shaft, the next step is engineering evaluation of the issue causing the rejection, which may result in acceptance or may require some action to correct a deficiency. Engineering evaluation and corrective measures are covered in subsequent sections of this chapter.

17.2.2 Evaluation Based on NDT

The non-destructive testing methods described in Chapter 16, including CSL, TIP and GGL are considered routine quality assurance tests for drilled shafts used in transportation infrastructure. The cost of all three tests is roughly comparable, and in most cases does not represent a significant increase to the cost of drilled shaft installation. The valuable information provided by these tests, namely verification of concrete integrity, easily justifies the cost of testing.

The approach recommended in this manual, and one that has been used successfully by many transportation agencies in the U.S., is to require NDT on all drilled shafts constructed using methods that involve placement of concrete under slurry or water ("wet" methods of construction) and on all load-test and technique shafts. The philosophy is that placement of concrete under slurry or water is more challenging and makes direct visual inspection difficult or impossible, thus creating greater uncertainty about concrete integrity. The risk associated with this higher level of uncertainty is addressed, in part, by requiring NDT methods. Integrity testing might also be considered for drilled shafts constructed by the dry method or with casing if there is concern regarding stability of the shaft excavation or any condition that poses a challenge to successful concrete placement (for example, a congested rebar cage).

The relative advantages and limitations of each test are covered in Chapter 16. CSL offers the potential to perform cross-hole tomography (CT) testing to further evaluate CSL anomalies, which can provide enhanced evaluation of anomalous zones and their locations within the cross section. TIP is currently the only method that provides assessment of concrete in the entire cross section. Some agencies combine methods, for example GGL and CSL, to assess concrete both within the core (CSL) and around the outside of the shaft (GGL). Some agencies now using TIP also require CSL for enhanced integrity evaluation and for comparisons between the two methods.

Another approach taken by some agencies is to require all drilled shafts to be installed with access tubes, but to perform CSL or GGL tests only when a question arises about concrete integrity in response to some observation or incident during construction. Alternatively, a specified percentage of drilled shafts can be tested as part of routine quality assurance, with the option of additional tests if anomalies are detected or in response to construction observations. This approach provides the opportunity for tests that utilize access tubes (CSL, GGL and CT) in any shaft and for any reason deemed appropriate by the engineer or contractor following construction. Whatever approach is adopted, it is important that NDT be integrated

into an overall philosophy that includes best practices for construction and inspection.

After construction, NDT results are reviewed by the responsible engineer for the presence of anomalies. Readings considered anomalous depend on the test method (see Chapter 16) and are summarized in Table 17-1.

TABLE 17-1 ANOMALOUS READINGS BY NDT METHOD

NDT Method	Parameter Measured	Class A Acceptable	Class B Conditionally Acceptable	Class C Abnormal
CSL	FAT Increase(%)	< 15%	see Table 16-2	see Table 16-2
	Energy Reduction, Decibals (dB)	≤ 9 dB		
TIP	Radius Reduction, R_r (%)	≤ 6%	-----	> 6%
	Local Cover (in)	≥ 3 in	-----	< 3 in
GGL	Standard Deviations below Mean	< 3	-----	≥ 3

If the NDT results are satisfactory and no anomalous readings are obtained, and if the inspection records are acceptable, the drilled shaft can be approved.

If CSL identifies an anomaly characterized as Class B according to Table 17-1 or Table 16-2, the inspection records are carefully reviewed particularly if the values are near the upper end of the limit. A significant defect would be indicated if it corresponds to an observation during construction that would also support the possible defect. When NDT results fall into Class C, with or without a correlation to an observed problem during construction, there is a need for further evaluation. The next step is often to perform another test method, or commonly cross-hole tomography (CT), which is used to verify the CSL anomaly and to better establish the limits of the anomalous zone within the cross section and vertically. Refer to the recommendations in the CSL Rating Criteria of Chapter 16 when considering CSL ratings of Class B and C.

When any of the three methods yields results falling into the Class C category of Table 17-1, further consideration of potential defects is warranted. Engineering judgment should be applied carefully before deciding whether concrete coring is needed; coring should not automatically be the next step. For anomalous zones limited to small areas of the shaft, and/or non-critical locations, the engineer may determine that the risk posed by a minor defect does not warrant the cost and effort of concrete coring. If further evaluation leads to the conclusion that coring is not warranted, the engineer's rationale for accepting the drilled shaft with anomalous NDT results should be documented in writing and made part of the permanent record.

If coring and laboratory strength testing of core samples are performed and show competent concrete with acceptable strength, the drilled shaft may be accepted. Factors that should be taken into account in evaluating acceptance include the diameter of the shaft, the number of core holes and their locations in the cross section relative to the location of the anomalous concrete. It is not uncommon to discover continuous, competent concrete in drilled shafts that exhibit CSL and GGL anomalies. If only one core boring was performed, consideration should be given to drilling additional core boring(s) to verify that a

significant defect was not missed by the initial core boring. When coring encounters sound or acceptable concrete conditions, and all other criteria are met, the drilled shaft should be accepted.

In the event that coring identifies zones of defective concrete, the next step is engineering evaluation of the defect for its impact on the performance of the shaft, and to determine the need for remediation. The remainder of this chapter provides descriptions of some of the types of deficiencies which might be encountered (Sections 17.3 and 17.4), evaluation methods used to assess the potential impacts of various deficiencies (Section 17.5), and remediation methods which can be employed to correct deficiencies (Section 17.6).

Figure 17-2 presents a more detailed subset of the lower two blocks shown in Figure 17-1, labeled 'engineering evaluation' and 'remediation.' The steps shown in Figure 17-2 are intended to provide a framework for evaluating the effects of defects on the ability of a drilled shaft to meet the LRFD design and performance criteria and, if necessary, to develop and execute an appropriate remediation plan.

There can be incidents during construction that are most effectively addressed prior to completion of concrete placement. Some examples are described below.

- *Sidewall sloughing of the hole prior to completion*. The obvious remedy is that the stability of the hole must be restored with one of the techniques outlined in Chapter 4. If the geotechnical strength of the shaft may be affected adversely, it may be possible to adjust the diameter or length of the shaft prior to completion.
- *Loosening of the ground around or below the hole such that the geotechnical strength is reduced*. Changes to installation methods, better control of drilling fluid properties or level, or adjustments in drilled shaft diameter or length prior to completion of the shaft may be a simple solution.
- *Inflow of water into a "dry hole" which might adversely affect geotechnical strength and/or concrete placement*. If the inflow cannot be controlled, then it is usually necessary to flood the excavation and to complete the shaft using wet hole techniques.
- *Problems with tremie concrete operations*. These can include failure of the joints in a segmental tremie, failure of the tremie to seal, withdrawal of the tremie from the concrete during placement, or a loss of workability of the concrete such that the tremie embedment cannot be maintained. In many (but not all) of these situations, the least expensive remediation strategy may be to stop the concrete placement, excavate the fluid concrete before it hardens, withdraw the reinforcement, and restart the process another day. Fluid or nearly fluid concrete can often be removed using a bucket or auger that will fit inside the reinforcing cage if there is so much concrete in the hole that the cage cannot be removed first. After extraction of the cage, the remainder of the concrete can be removed and the hole may need to be deepened slightly to restore a clean base or to compensate for any loosening of the ground which may occur.

  In general, any decision to abandon concrete placement and re-excavate the hole is made by the contractor, who has the contractual responsibility to complete the work. The inspector's role is to observe operations and to report conditions which may be noncompliant with the specifications and/or installation plan, or that might otherwise impair the performance of the completed drilled shaft, to the project superintendent, construction manager, and other appropriate representatives in a timely manner.

  It might also be noted that a problem with tremie concrete operations can be effectively addressed by suspending concrete placement with plans for a subsequent cold joint and completion of the concrete placement in the dry. This approach is generally only suitable if the concrete level has achieved an elevation that is above the groundwater or within a permanent casing.
- *Sidewall sloughing or water inflow during concrete placement*. If these problems occur during

placement of concrete, the best strategy is likely to be to abandon the concrete placement operations and re-excavate the hole as described above.

The most effective stage of the project to discuss and evaluate potential construction and concrete placement problems and to develop contingency plans is prior to the start of construction. Contingency plans are appropriate requirements for submittal as a part of the Drilled Shaft Installation Plan and discussed during pre-construction coordination meetings attended by the contractor, engineer and inspection personnel.

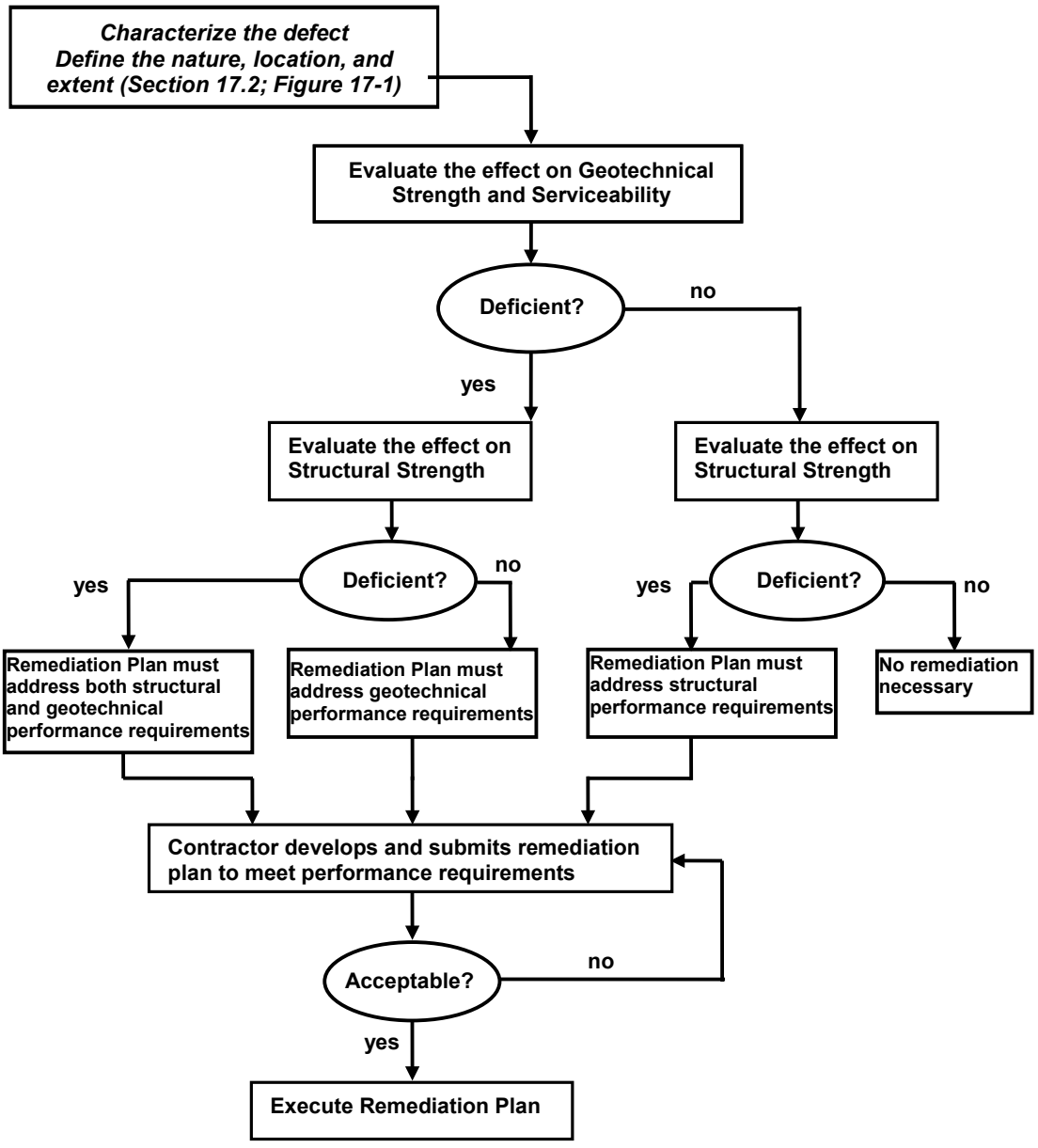


Figure 17-2 Flow Chart of the Evaluation and Remediation Process

17.3 TYPES OF DEFICIENCIES IN COMPLETED SHAFTS

The remainder of this chapter deals with deficiencies and remediation in completed drilled shafts. The defects described below may require remediation if the magnitude and location is sufficient to affect the strength or serviceability of the drilled shaft.

17.3.1 Geotechnical Strength or Serviceability

Even though the shaft may be structurally sound, there can be deficiencies with respect to geotechnical strength or serviceability. These include incorrect dimensions or location, and construction problems which lead to inadequate axial or lateral resistance. Some deficiencies may be identified with existing structures due to a change in ground conditions or loadings for which the foundations were not originally designed.

In many cases the design can be made more robust and accommodating of disturbance from construction activities by a conservative design with respect to axial or lateral resistance of shallow soils. A sensitivity analysis during design may indicate that a cost-effective design can be produced with relatively little reliance on shallow strata, thus avoiding or minimizing potential issues relating to axial side resistance. However, the cost tradeoff of being overly conservative with respect to lateral resistance at shallow depths can be significant, so engineering judgment is required.

17.3.1.1 Incorrect Dimension or Location

Incorrect dimension or length may result from simple errors or inattention to the geotechnical conditions actually encountered. For instance, a drilled shaft has an anticipated tip elevation of +75 ft and is also required to penetrate 30 ft into a shale formation that is anticipated to be encountered below elevation +105 ft. If the top of rock is encountered at an elevation which is deeper than anticipated, then the shaft must be lengthened accordingly to provide the 30-ft socket and satisfy the geotechnical strength requirements. Failure to deepen the shaft in accordance with the actual ground conditions may result in a reduced axial resistance.

A drilled shaft which is constructed at a location out of position as a result of a survey error may be subject to unanticipated eccentricity.

17.3.1.2 Inadequate Base Resistance

Inadequate base resistance can result from several factors such as:

- Weak soil at the base because the shaft was founded in the wrong stratum.
- Weak soil at the base because of inadequate cleaning, or accumulation of sediments from drilling slurry prior to concrete placement, leaving loose debris between the concrete and the bearing material.
- Loosening of the bearing stratum due to instability at the bottom of the excavation prior to concrete placement.
- Weak concrete or segregated aggregate at the base because the concrete was dropped through water within the shaft excavation.
- Subsidence or loss of bearing at the drilled shaft foundation. Construction activities which could affect base resistance include vibrations from nearby construction operations, such as pile driving, and construction of deeper drilled shafts in very close proximity. In karst terrain or mining areas,

a previously undetected void might be identified beneath an existing drilled shaft.

17.3.1.3 Inadequate Axial Side Resistance

Inadequate axial side resistance can result from several factors such as:

- Failure to recover temporary casing within a zone contributing to side resistance.
- Softening of the soil or rock at the sidewall due to excessive exposure or seepage. Some types of shales and claystones are susceptible to rapid weathering in an open excavation.
- Excessive buildup of contamination at the sidewall due to excessive exposure to bentonite slurry.
- Loosening of the soil around the shaft due to instability of the excavation.
- Settlement of soils due to adjacent embankment construction or material stockpiles.

17.3.1.4 Inadequate Lateral Resistance Around Shaft

Inadequate lateral resistance around the drilled shaft can result from failure to recover temporary casing which was installed in an oversized hole, or due to collapse of the excavation and disturbance of surrounding soil during construction. Other construction operations such as installation and extraction of temporary piles can also affect soil contributing to lateral resistance.

17.3.2 Structural Strength

Even if the drilled shaft is constructed adequately with respect to geotechnical strength, deficiencies relating to inadequate structural strength can occur. Structural demands on drilled shafts loaded primarily in axial compression are usually low, and so the greatest concerns arise when there are structural defects within the length of the shaft subject to significant bending moments. Some types of structural defects which may require evaluation and remediation are discussed below.

17.3.2.1 Incorrect Dimensions or Location

Errors in which the shaft is constructed in the incorrect location or with incorrect dimensions or reinforcing may pose an issue requiring evaluation of structural performance.

17.3.2.2 Structural Defects in Concrete

Structural defects in concrete are among the most commonly encountered problems with drilled shaft construction. One likely reason that defects in concrete are so often noticed is because the integrity testing methods described in Chapter 16 have become so effective that even minor flaws not previously noticed are now identified. Another is the fact that drilled shaft foundations are increasingly used to large diameters and great embedded length with very congested reinforcing cages, often with embedded column reinforcement. As a result of the industry's success in completing such difficult projects, the constructability demands on the concrete placement operations are more severe than was ever previously attempted. Designers should consider the issues and concerns outlined in Chapters 6 and 7 so that reinforcing cages and concrete are appropriately designed for constructability to reduce the risks of structural defects.

Some of the problems resulting in structural defects in concrete include the following:

- Soil inclusions due to sloughing of the sidewall during concrete placement.

- Soil inclusions due to settlement of suspended solids in the drilling fluid during concrete placement. Slurry materials and testing are described in Chapter 7.
- Entrapped laitance due to excessive bleeding, segregation, and/or loss of workability in the concrete. Concrete properties and the importance of workability are described in Chapter 5.
- Cold joints or trapped laitance or debris due to an interruption in the tremie placement of concrete under water or slurry.
- A breach of the tremie seal into the fresh concrete during placement due to improper withdrawal of the tremie or a sudden drop in the concrete level such as might be associated with flow into voids around the excavation as casing is withdrawn.
- Leaching of the concrete due to inflow of water through joints or voids in the subsurface formation. Artesian groundwater conditions require that a positive pressure head be maintained in the shaft excavation at all times.
- Necking or displacement of concrete due to arching within a temporary casing as it is withdrawn.
- Infiltration of drilling fluid into joints of the tremie pipe.
- Inadvertent use of incorrect concrete mix of lower strength, poor concrete workability, or failure to maintain minimum slump for the duration of concrete placement.

17.3.2.3 Inadequate Cover or Exposed Reinforcement

Some of the structural defects in the concrete described above can compromise the cover of concrete around the reinforcement. Such defects can occur from trapped debris or laitance or from loss of workability during the placement operation that leads to blockage and failure of the concrete to flow through the cage. Structural reinforcement which is or may be exposed to corrosion within the life of the structure represents a deficiency which may require remediation.

17.3.2.4 Structurally Damaged Shaft Due to Overload after Construction

A drilled shaft can be damaged and in need of remediation if it is overloaded to the point of structural yield. Existing structures may be affected by vessel or truck impact loads in excess of the structural strength of the shaft. The authors are aware of cases in which a drilled shaft has been impacted by a contractor work barge or runaway equipment during construction, resulting in structural damage to a column/shaft that required remediation. In some cases of this type, an accurate and reliable assessment of the current condition of the foundation may present a significant challenge; if the strength of the damaged foundation cannot be assured, then a conservative assessment must be made for purposes of remediation.

17.4 EVALUATION

A full restoration of the drilled shaft to its design structural and/or geotechnical strength may be accomplished in some cases without the need for anything more than a cursory engineering evaluation and modest repair program. In general, this approach is used where structural defects (see Figure 17-2) may be close to the top of the shaft and easy to repair by simple hand methods of chipping and reforming the shaft. For instance, Caltrans standardized mitigation plans allow that the contractor may choose to implement one of the standardized mitigation plans to restore the full strength of the shaft without further need for evaluation. For defects which are less easy to repair, an engineering evaluation is the first step toward determining the need for remediation and subsequent development of a remediation plan to correct any deficiency.

The evaluation of geotechnical strength involves an assessment of the nature and extent of the problem and the potential impact on the geotechnical performance. In cases where only the geotechnical strength is in question, it may be possible to consider the following actions:

- Evaluate the actual demands on the specific shaft in question. For simplicity during design, often all of the shafts in a group or within a section of the project are designed for the largest load demand which is anticipated for any shaft; the specific shaft in question may be subject to an actual load demand that is somewhat less.
- Evaluate the specific geotechnical conditions at the location of the shaft. This evaluation may involve additional geotechnical exploration if a boring at the specific shaft location is not available. For example, it may be found that the soil or rock conditions at the specific shaft in question may be more favorable than the general simplified conditions used to establish tip elevations. If so, it might be possible to justify an increase in the estimated axial resistance in some portion of the shaft to compensate for a deficiency elsewhere (such as a stuck temporary casing, for instance).
- Perform a proof test of the shaft in question. In some cases, an evaluation of geotechnical strength could include a proof test using a rapid load test, dynamic load test, or even a static load test. Even if the geotechnical strength is determined to be less than the nominal strength used for design, the fact that a proof test was performed on this specific shaft should allow a higher resistance factor to be used for this specific shaft. The specific resistance factor to apply to an individual drilled shaft subjected to a proof test is not defined by AASHTO LRFD Specifications at present and would require the judgment of the design engineers.

The evaluation of structural strength involves an assessment of the nature, location and extent of the structural defect and the potential impact on structural performance. Structural considerations are generally dominated by flexural strength demands. A p-y computer model of the shaft using the methods outlined in Chapters 9 and 12, using the actual load demands and ground conditions for the specific shaft in question, can determine the flexural demand at the location of the structural defect. Often, the bending moment is small in the deeper portions of the shaft.

If the defect is within a portion of the shaft that includes permanent casing, the actual as-built structural strength of the composite section may be considered in this portion of the shaft. If the defect is assessed to consist of low strength concrete, a steel shell section filled with lower strength reinforced concrete might still provide sufficient structural strength in some cases without implementing remediation measures.

Evaluation of the structural strength of a drilled shaft cross section with low strength concrete or a soil inclusion over a portion of the section should be based on a conservative assessment of the character and extent of the defect from integrity testing, coring, and/or other techniques. Exposed reinforcement may be discounted because of possible buckling of the bars or future corrosion. A discussion of the flexural behavior of drilled shafts with minor flaws is provided by Sarhan and O'Neill (2002). The structural evaluation of the strength of an imperfect cross section requires engineering judgment and analyses that may be beyond the scope of routine design work. However, advanced analyses may be warranted in cases where a rigorous treatment provides an engineering basis for making repair decisions and for evaluating repair options.

An analytical approach to evaluating the structural strength of drilled shafts with defects is described by Turner and Turner (2016). The approach is based on finite element analysis using a 'fiber model' of the reinforced concrete section, in which the cross section is discretized into separate zones (fibers) each characterized by unique uniaxial stress-strain behavior representative of the reinforcing steel, confined

concrete, unconfined concrete, and/or defective concrete. This analysis allows the complex nonlinear stress-strain behavior of each material to be handled independently and exactly. To illustrate, the authors present a case history in which drilled shafts found to have defective concrete were remediated by removing the defective concrete via hydro-jetting through drilled access tubes and replaced with high-strength grout. However, post-jetting inspection by video camera showed that the jets were unable to remove the defective concrete from a "shadow" zone located behind each of the longitudinal reinforcement bars. Because the defective concrete remains in these zones, there was concern that the moment resistance of the section is reduced (by an unknown amount) compared to the case where all of the defective concrete had been replaced, and initially rejected the repair. Figure 17-3 shows the as-designed cross section and the discretized fiber model of the as-repaired section. The shadow zones are modeled in this case as having zero strength. The fiber model analysis was used to calculate the nominal moment capacity through a moment-curvature analysis that made explicit consideration of the remaining defective concrete. The results, shown in Figure 17-4, show that, despite the slight decrease in structural resistance attributable to the defects, the section was still capable of resisting the factored demands of the project, and the repair was accepted. This type of analysis is possible using commercially available software.

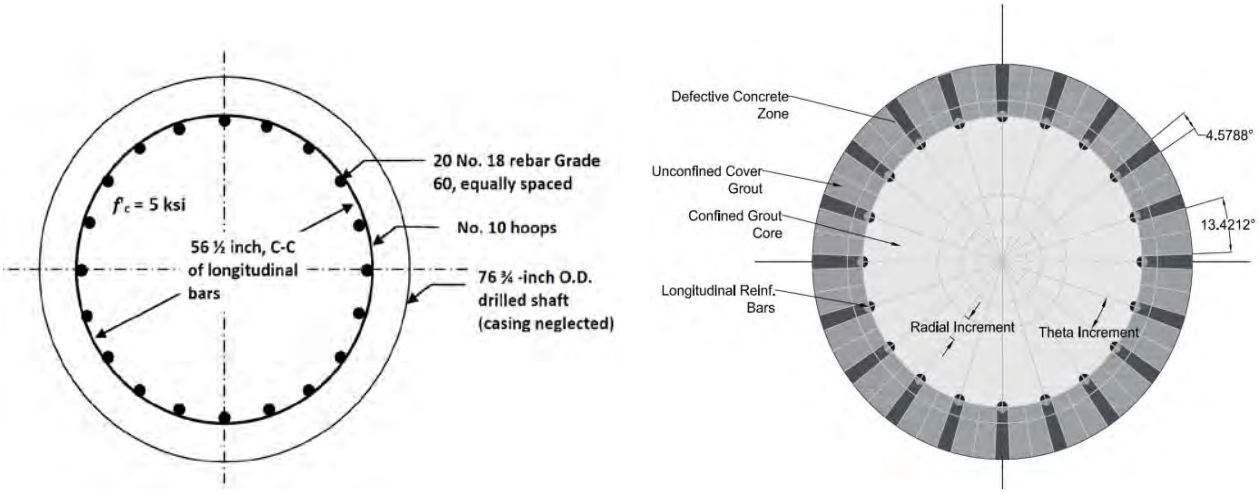


Figure 17-3 (a) As-Designed Drilled Shaft Cross Section, (b) Fiber Model of Repaired Shaft (Turner and Turner, 2016)

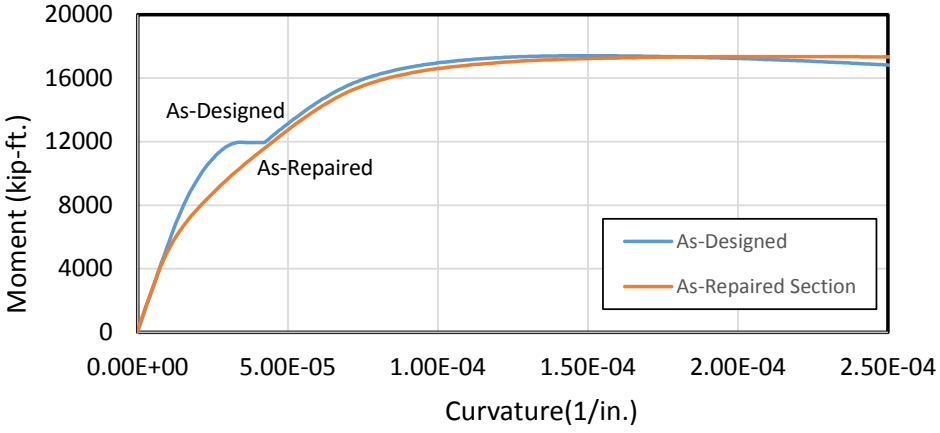


Figure 17-4 Moment-Curvature Analysis Results for As-Designed and As-Repaired Sections (Turner and Turner, 2016)

17.5 REMEDIATION METHODS

After evaluation of the defect and determination of a deficiency in geotechnical or structural strength, a plan for remediation is required. The plan should provide a means to restore the required performance characteristics of the drilled shaft in a way that is constructible and reliable, and that includes a program of monitoring or verification that will provide assurance that the remediation is successful. The sections which follow outline some remediation measures that have been used successfully. The list is not exhaustive, but is intended to provide a useful resource.

17.5.1 Ground Improvement

Ground improvement techniques can be used to restore geotechnical strength for situations where disturbance of existing materials may have occurred during construction or due to environmental factors. Ground improvement techniques might include:

- Penetration grouting to fill voids around a shaft such as might occur with a casing left in place in an oversized hole.
- Compaction grouting to restore density of granular soils that may have been loosened below or around a drilled shaft.
- Vibro-compacted stone columns to improve density and reduce liquefaction susceptibility around an existing drilled shaft.
- Grouting might also be employed to treat loose materials below the base of a shaft by drilling through the shaft to access the bearing soils below the base. This type of grouting operation is more of a replacement technique than a ground improvement, and is described subsequently in Section 17.5.5.1.

In some cases, ground improvement of shallow soils to improve stiffness and strength around foundations for lateral loads may consist of excavation of loose or disturbed soils and replacement with flowable fill. Flowable fill is a cement-stabilized sand mixture that can be placed like fluid concrete without the need for compaction. Other applicable techniques might include deep soil mixing, jet grouting, lime stabilization, and vibratory compacted stone columns, among others. Rollins et al. (2010) provide recommendations for analysis and design of ground improvement techniques to improve the lateral performance of deep foundations.

17.5.2 Supplemental Foundations and/or Structural Bridging

In some cases, where the strength or stiffness of a drilled shaft is less than required, the most effective remediation strategy might be to add additional deep foundation elements. These might be designed to supplement or even completely replace the existing shaft. Additional shafts or micropiles are generally the most suitable type of deep foundation support element for use near an existing shaft, as described in more detail below.

It should be noted that the design of a foundation incorporating additional deep foundation elements into a common cap with the existing drilled shaft must address the issue of strain compatibility; the shear distribution in the cap will be affected by the relative stiffness of the various elements supporting the cap. For instance, if additional deep foundations extend into rock and are very stiff in comparison to an existing shaft, the cap may be subject to high shear forces and bending moments as illustrated in Figure 17-5. Additional deep foundations composed of relatively flexible micropiles may be relatively less stiff in comparison to a large diameter existing drilled shaft foundation. Pre-loading of the new foundation elements can sometimes be used to address strain compatibility.

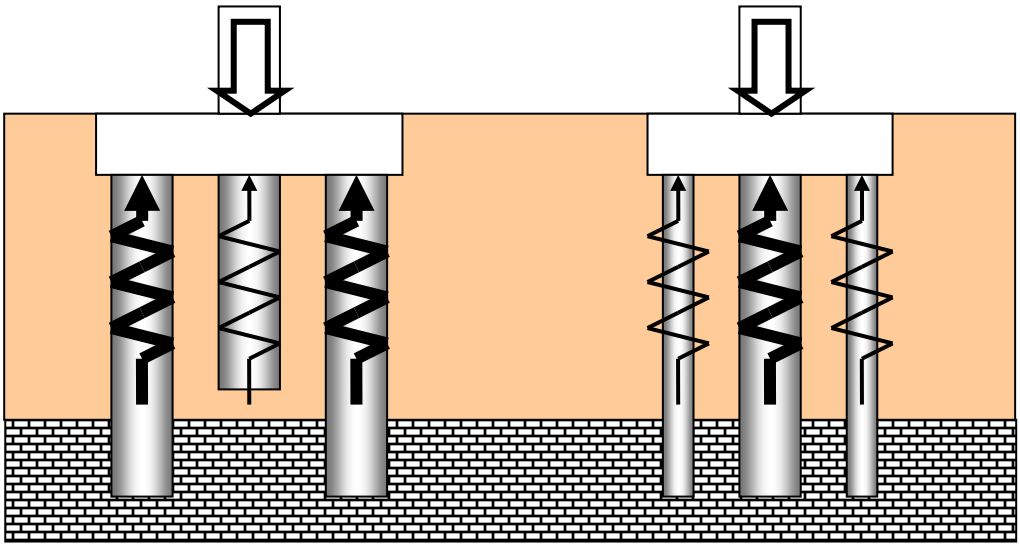


Figure 17-5 Differing Relative Stiffness of Deep Foundation Elements Can Affect Shear Forces and Bending Moment in Cap

17.5.2.1 Additional Shafts

Underpinning with "straddle shafts", sometimes called "sister shafts," is a method of supplementing or replacing a defective drilled shaft. The existing shaft may be chipped away and eliminated from inclusion in the new foundation, but this approach is usually not necessary unless the existing shaft is so badly damaged that it is considered a liability and of no benefit.

An example of the use of straddle shafts is illustrated in Figure 17-6, where existing pile foundations for a bridge in Arizona were completely undermined by scour. These foundations represented a complete replacement of the existing foundation.



Figure 17-6 Use of Straddle Shafts to Replace an Existing Foundation (photo by B. Berkovitz)

The sketch and photo in Figure 17-7 illustrates another example of the use of "sister shafts" to provide supplemental support for an existing drilled shaft which was structurally sound but deficient in axial resistance. An additional 4-ft diameter shaft was constructed on each side of the existing column, which itself was supported on a single 6-ft diameter shaft. To keep the size of the cap as small as possible, the "sister shafts" were constructed as closely as possible and the composite foundation was analyzed as an elliptically shaped barrette foundation with end bearing contributions only from the two new, deeper shafts. The column was roughened and tied to the new post-tensioned cap using dowels.

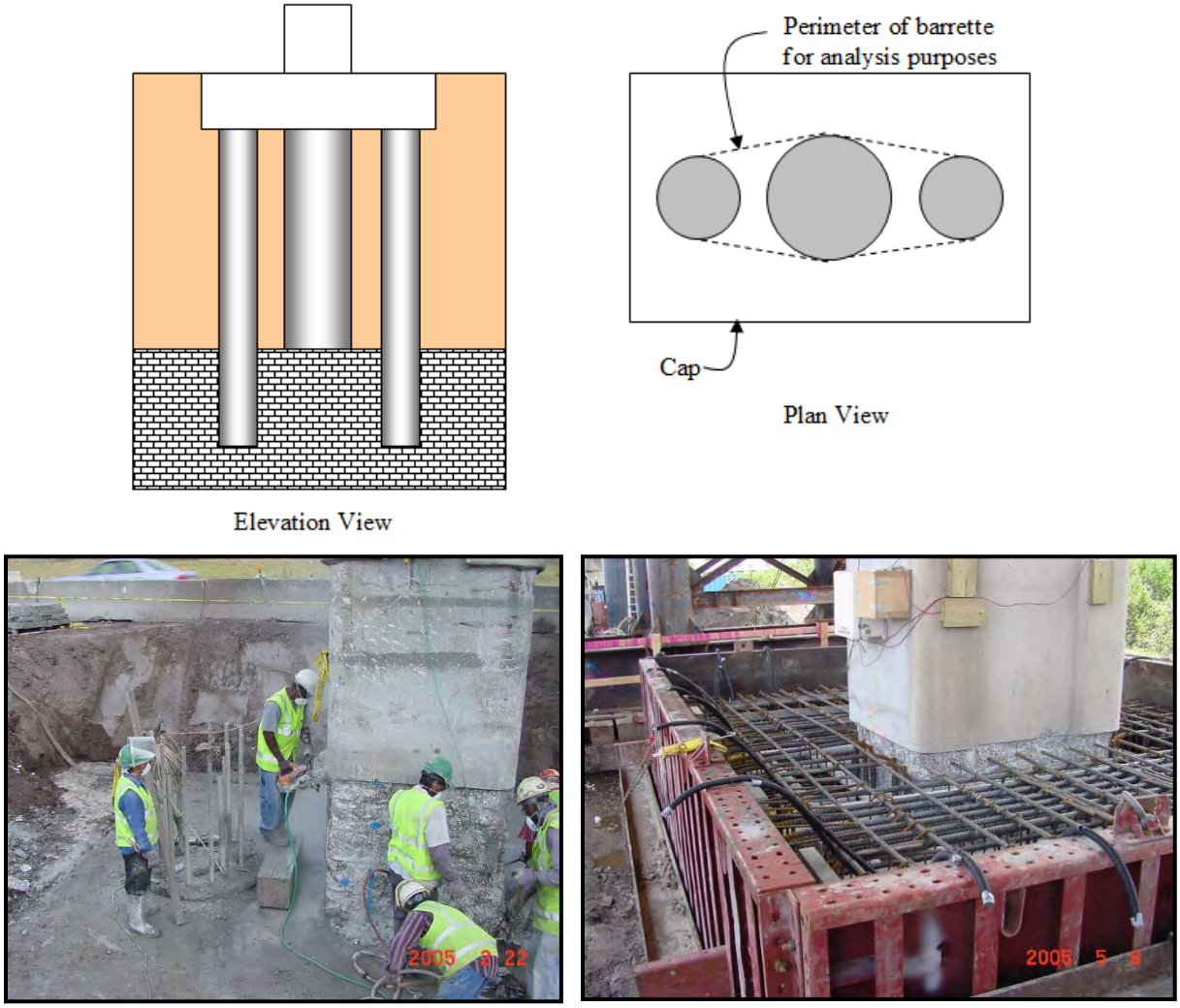


Figure 17-7 Use of Sister Shafts to Supplement an Existing Foundation (McGillivray and Brown, 2007)

17.5.2.2 Micropiles

Micropiles may be used as additional deep foundation elements to supplement or replace an existing shaft. Micropiles are typically 12 inches or smaller in diameter, and are constructed as a steel member (pipe or high strength bar) which is grouted into the bearing material. The design and construction of micropiles is described in FHWA-NHI-05-039 by Sabatini, et al. (2005). One advantage of micropiles for underpinning work is the small size of the equipment used to install these foundations, which often allows

their use in locations with restricted access or severe height limitations. Sometimes micropiles can even be installed by drilling through an existing drilled shaft to anchor the shaft into an underlying formation, as illustrated in Figure 17-8. For this application, the top of the shaft must be accessible. In some cases, it is even possible to install these by drilling through existing tubes used for integrity testing. High strength bars or even conventional reinforcing bars, depending upon the application and loads, may be used to form the micropile. The micropile steel must have sufficient development length to bond to the existing shaft. When installed through the existing drilled shaft, micropiles can provide additional structural strength to the shaft.

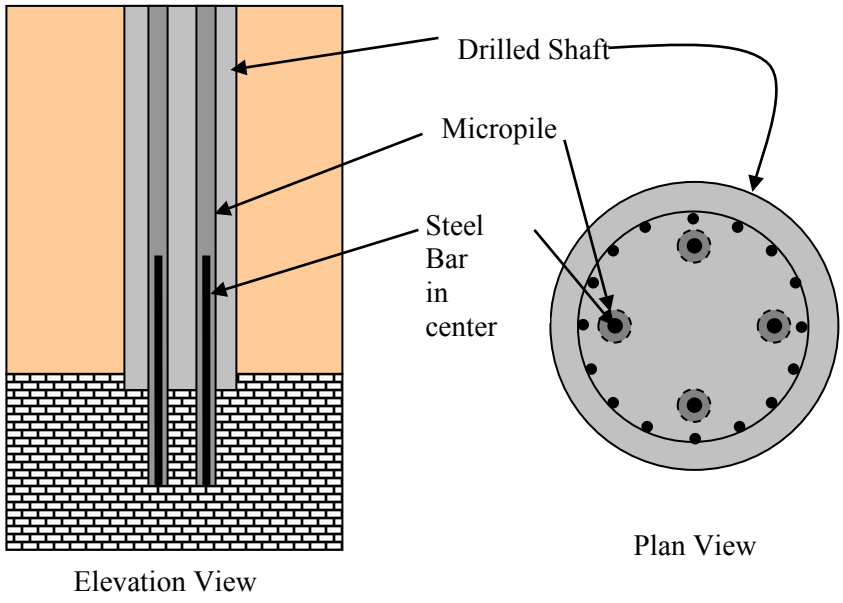


Figure 17-8 Use of Micropiles Drilled Through Existing Drilled Shaft

Where micropiles are used around an existing shaft, as illustrated in Figure 17-9, it is frequently possible to construct the piles very close to the existing shaft and thereby minimize the size of the cap. These micropiles are constructed using 10.75-inch O.D. steel pipe casing installed to rock, with a 75 ksi grouted bar installed within the casing to form a socket into bedrock. A cap was subsequently formed around the column and micropiles similar to the one shown in Figure 17-7.

17.5.3 Excavation and Replacement

Excavation and replacement is a feasible repair method for structural defects in shafts that are relatively shallow and therefore accessible from the surface. Poor quality concrete in the top of a large diameter shaft may be removed by hand with impact tools, and it may be possible to use a drilled shaft rig to more efficiently excavate concrete within the reinforcing cage with rotary tools. If the upper portion of the soil around shaft can be excavated and a dry working environment secured, hand methods may be used to chip away defective concrete from the outside of the shaft, and the upper portion of the shaft re-cast within a form. Figure 17-10 provides two photographs of this type of repair work.



Figure 17-9 Use of Micropiles to Supplement an Existing Foundation (McGillivray and Brown, 2007)



(a) (b)

Figure 17-10 Excavation and Replacement of Defective Concrete Using (a) Drilling Tool within the Cage and (b) Hand Methods (photos courtesy Caltrans)

17.5.4 Structural Enhancement

Structural enhancement can be performed to increase the structural strength of a defective drilled shaft without complete removal of the defect. For instance, if the center of the shaft is drilled out as illustrated in Figure 17-10a, it may be possible to install a structural steel or pipe section cast into the central portion of the shaft with high strength concrete. Figure 17-11 shows plan drawings for a repair in which a major zone of defective concrete in the core of the shaft was first excavated to a depth several feet beyond the defective zone. A 30-inch diameter steel pipe was then centered and grouted into the excavated core of the shaft. Analysis of the repaired section demonstrated that the flexural resistance was equal to or greater than the as-designed section. This drilled shaft was used for support of an overhead sign structure and designed primarily for lateral loading due to wind. It may also be possible to extend a central drilled section into the bearing formation below the shaft to increase the geotechnical strength of the shaft.

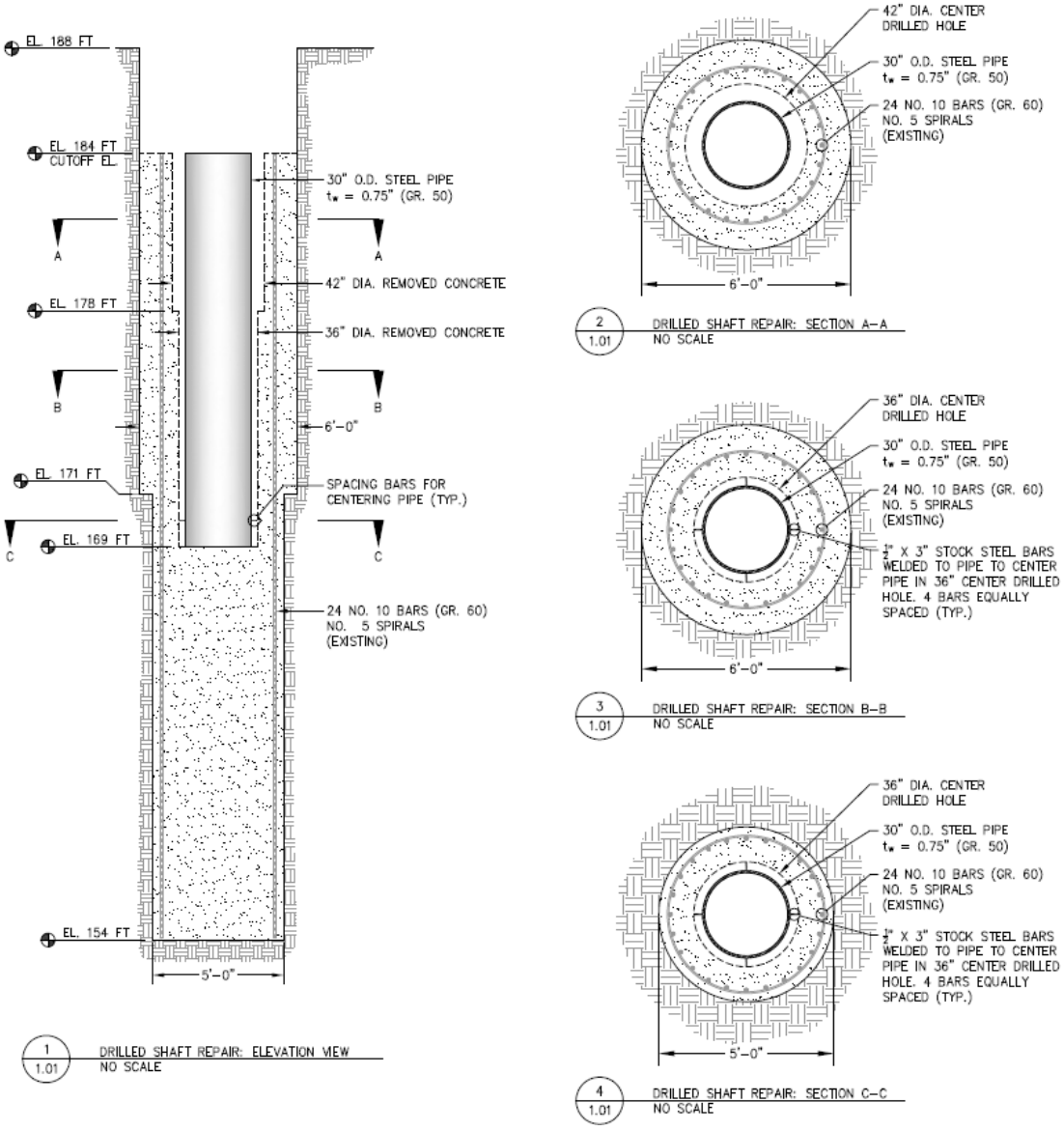


Figure 17-11 Use of a Steel Section Cast into a Drilled Shaft to Form Structural Bridge

Additional reinforcing bars can also be used to provide structural enhancement. The concept is to provide a structural 'bridge' capable of transmitting force demands from the non-defective concrete above the defective zone to non-defective concrete below the defect. Figure 17-12 shows an example in which seven No. 18 bars were used to bridge a zone of defective concrete in a 10-ft diameter drilled shaft. A set of seven 4-1/4 inch diameter holes were first drilled through the defective zone into the competent concrete below. In the target repair zone, each hole was then increased to a diameter of 2 ft using a hydrodemolition tool. A single No. 18 bar was then centered into each hole and the repair was completed by grouting the holes. In this case, the additional steel reinforcing in combination with the new grout in the 2-ft diameter holes provided the needed structural resistance to meet the load demand. Above and below the repair zone, each bar was centered and grouted into a 4-1/4 inch diameter hole over a length sufficient to meet development length requirements for a No. 18 bar.

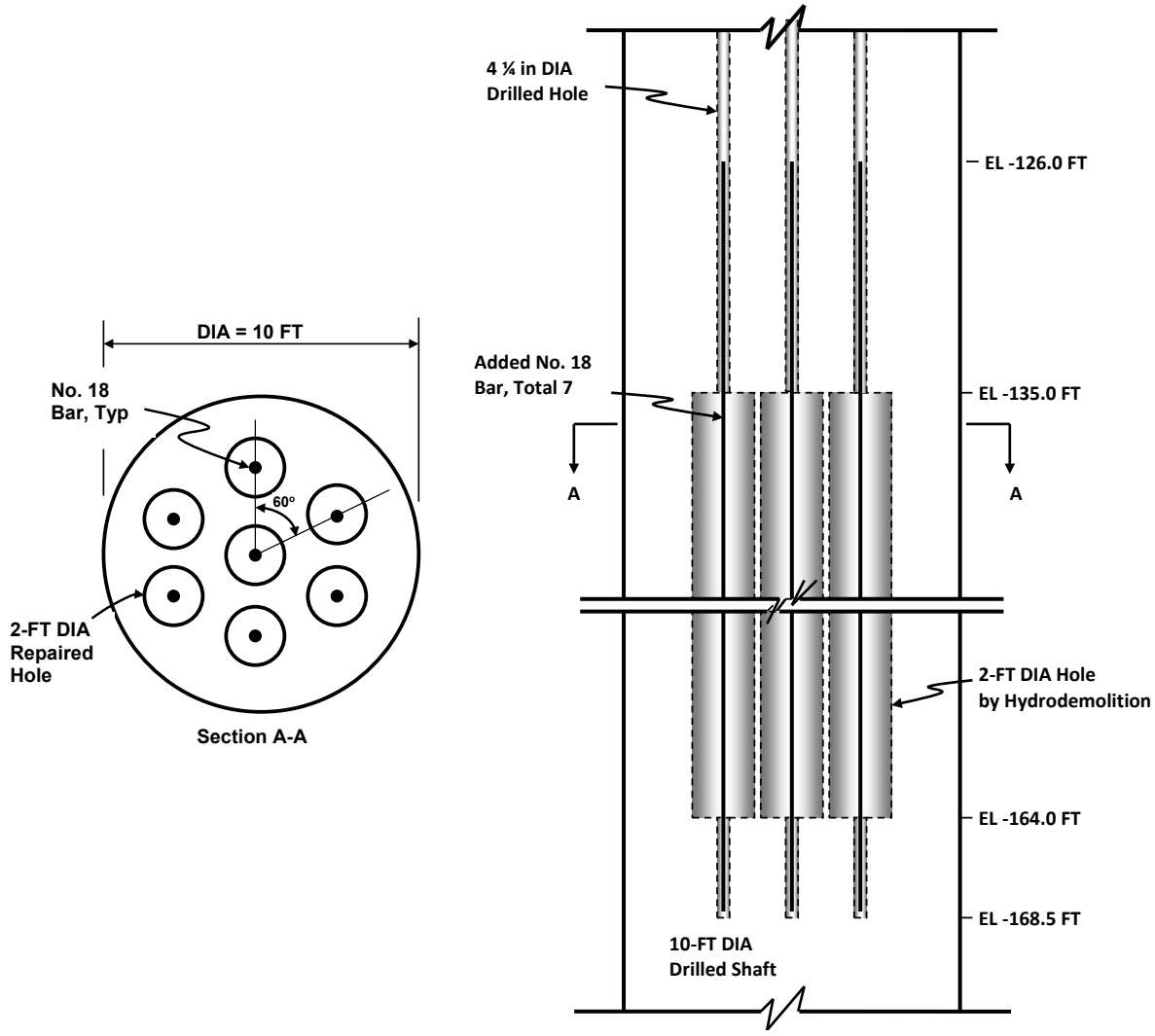


Figure 17-12 Structural Enhancement using Additional Reinforcing Bars

17.5.5 Grouting

Grouting may provide a means of directly treating the defective area to improve or restore the structural and/or geotechnical strength. Grouting may be performed at the base of the drilled shaft to address a problem of loose material below the shaft base, within the shaft to address structural defects, or around the shaft to provide bond to the soil and/or cover for corrosion protection.

17.5.5.1 Grouting at the Base of the Shaft

A defect at the shaft base, caused by trapped material or segregated aggregate, can sometimes be treated by grouting the base of the drilled shaft. The grouting of the base of a drilled shaft will usually involve the following steps:

- At least two holes are drilled through the full length of the shaft so that fluid can be circulated to and through the soft zone at the base of the drilled shaft. These conduits are normally available as access tubes if access tubes have been installed for integrity testing or as drill or core holes that

were placed during an investigation of the quality of the base.

- Wash or flush the defective zone by pumping water or an air - water mixture down one tube and returning it with suspended debris through the other(s).
- After a clear return is obtained and debris has been removed to the extent possible, inject the grout into one hole until clean grout returns through the other(s). After grout has flushed all of the wash water through all of the tubes, seal all of the return tubes and apply pressure grouting.
- Monitor the shaft for upward movement, and log the volume and pressure of grout taken as a function of time as would be performed with planned base grouting for enhancement as described in Appendix H.

With at least two holes through the drilled shaft and into the weak base material, the weak material at the base can be washed away by forcing fluid down one of the holes and having it return through the other. An air-water mixture under high pressure can be an effective technique except when the drilled shaft is founded in cohesionless material. In that case, no air and low water pressures must be used so as not to undermine the shaft being treated or nearby drilled shafts. The solids in the returning fluid should be monitored during the process as a means of evaluating the efficiency of the washing operation. As the cleaning of the base of the drilled shaft progresses, it may be possible or desirable to inspect using a television camera or borescope if the base is within a rock formation.

17.5.5.2 Grouting Within the Shaft

Grouting may be used to repair zones of defective concrete within a drilled shaft. In this application, the most challenging part of the operation may be the removal of low strength material. Hydrodemolition, or erosion of weak concrete using high pressure water jets, has been used with success. Access holes are used to introduce water jets and observe communication between holes and the return up through these holes. Some photos of downhole hydroblasting equipment are provided in Figure 17-13, and a photo of the zone treated using the equipment shown in Figure 17-13**.** is shown in Figure 17-14(a). The photo was obtained using the downhole video camera shown in Figure 17-14(b) and was used to verify required removal of the defective concrete. Reinforcing bars and CSL access tubes can also be seen. The shaft and equipment depicted in these photos is described as a case history by Daita et al. (2018). These high pressure water jets are capable of nozzle pressures in excess of 40,000 psi and can cut concrete at close range (within a foot or so) if the jet can be directed and is not shadowed by steel reinforcing. It is not normally feasible to remove large quantities of concrete in this manner.

After completion of the hydrodemolition and water jetting to flush the cuttings and debris from the hole, the access hole should typically be pumped dry or vacuumed to provide the most effective means for grout to fill voids effectively. However, this may not be possible if there is seepage due to communication with groundwater outside the drilled shaft. The photo in Figure 17-15 shows a core taken through the grouted zone within a repaired shaft, with the grout stringers clearly visible within the concrete matrix.

This technique can be used to remediate and improve concrete which has inclusions of soil or low strength concrete, and often is sufficient to restore the structural strength to meet the project requirements. Grouting cannot be expected to restore the shaft to a perfect condition. Post-treatment cores or crosshole sonic logs should show improvement, but will not be free of anomalies.

If segregation of concrete aggregate is suspected from the construction records, and if the presence of relatively clean aggregate is confirmed by core holes and water flushing, treatment with permeation grouting may be a feasible option. The grout used for such applications would consist of microfine grout. The hole preparation for permeation grouting would be essentially the same as described above for the

hydrodemolition method. When the defective zone is sufficiently cleaned by flushing, grout would be pumped under pressure sequentially from the available core holes until clean grout returned from the return holes. As clean grout is observed, each return hole would be capped, and the grout pressure increased to maximize grout penetration. The effectiveness of the grouting can be assessed by CSL testing after the grout sets.



(a) Blasting Nozzle (b) Hydro Jet Platform

Figure 17-13 Hydrodemolition Equipment



(a) (b)

Figure 17-14 Video Inspection of Drilled Shaft Remediation: (a) Snapshot from Video Inspection, and (b) Downhole Video Camera

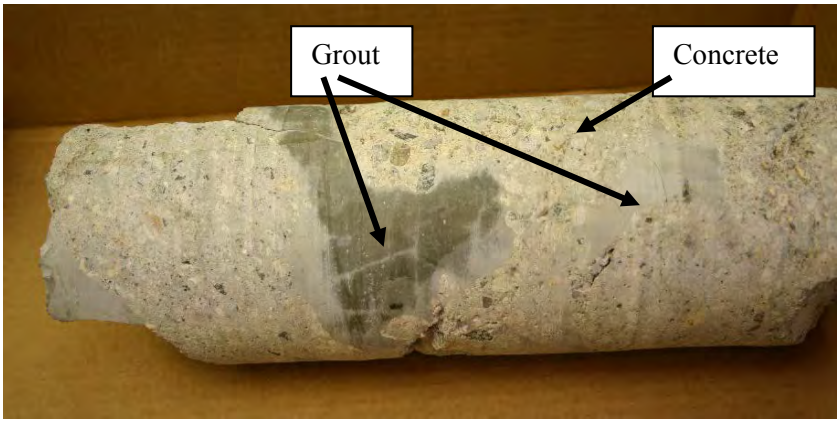


Figure 17-15 Concrete Core Through Grouted Concrete Repair (photo from Ryan McTigue, Raito, Inc.)

Grouting within the shaft as described above may not be effective if the defects to be treated include zones on the outside of the reinforcing cage in granular soils below the groundwater. In such a case, attempts to hydroblast outside the shaft would erode unstable soils which might be expected to cave. Jet grouting around the perimeter of the shaft is a technique which might be considered, as described in the following section.

17.5.5.3 Grouting Around the Perimeter of the Shaft

If the shaft is structurally sufficient except for concerns regarding the concrete cover on the reinforcement, or if a void exists between the outside of the shaft and the soil, then grouting around the perimeter may be considered. In the simple case of a stuck casing with a void left around the casing, tremie grout into the void using small diameter tubes may be sufficient to fill the gap and restore the lateral soil resistance. If there are voids or defects in the concrete around the perimeter of the shaft below the groundwater level, jet grouting may be considered as a means to erode soil or weak material and provide a cementitious encasement of the shaft.

Jet grouting has a history of use for ground improvement, often for stabilization of weak soils around excavations or walls. A column of grout is formed by a drill equipped with sideward directed nozzles which cut the soil and flush most of it back to the surface, replacing it with grout. The jets can include combination of grout, water and air to enhance the cutting ability, with capability of creating columns of grout from 3 ft to more than 7 ft in diameter depending on the type of flushing system used and the applied grouting paramters (pressure, rotation speed, grout monitor lifting rate, etc.). Photos of jet grouting equipment and some excavated jet-grout columns are provided in Figure 17-16 (Axtell and Stark, 2008).

The nozzle pressure is much lower than that used for hydrodemolition and typically around 1000 psi. This pressure will cut soil or relatively weakly cemented inclusions in the shaft but will not erode good concrete. Therefore, the technique is most useful for encapsulating a drilled shaft as illustrated in Figure 17-17. Note that the jet grout columns can be started and stopped at predetermined elevations corresponding to the zone needing treatment, and it is not necessary that the column extend the full height of the shaft. It is also possible that jet grouting could be used to treat an area around a shaft to stabilize the shaft and adjoining soil prior to internal hydrodemolition and grouting.



Figure 17-16 Jet Grouting Photos and Illustration

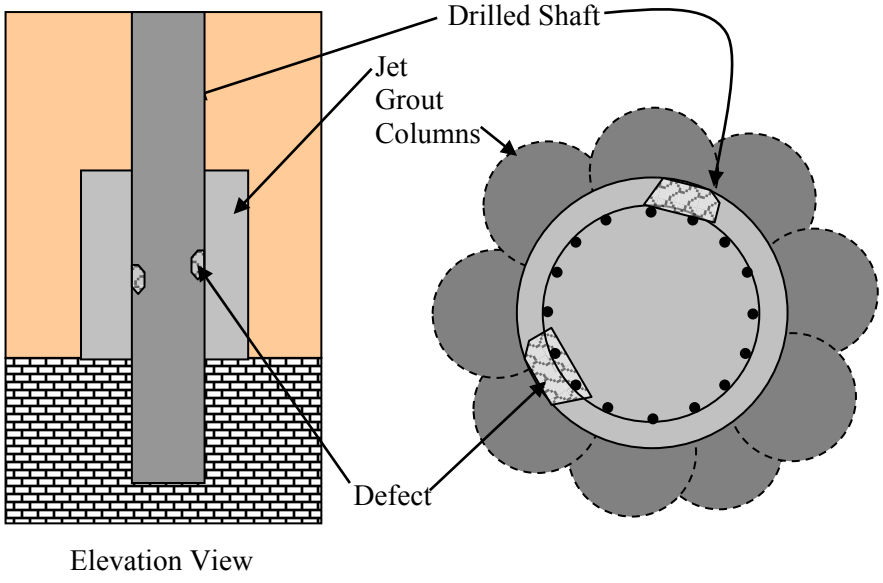


Figure 17-17 Jet Grouting Application for Drilled Shaft Repair

7.6 SUMMARY

This chapter outlines the process of assessment and acceptance of drilled shaft installations, and provides guidance on the types of information used to evaluate completed drilled shafts. Inspection records and NDT methods provide the basis for establishing whether the completed drilled shafts meet the project objectives, as defined by the Plans and Specifications. In cases where construction records and/or nondestructive testing (NDT) suggest the possibility of defects, guidance is provided on the follow-up actions needed to further evaluate the nature and extent of potential defects and to assess their effect, if any, on the ability of the as-built drilled shaft to meet the design criteria for strength and serviceability. In cases where it is determined that a drilled shaft is not adequate, methods for remediation and repair are presented.

An overview is presented of some of the types of deficiencies which can be encountered in drilled shaft construction. Strategies which might be effective in restoring the shaft to meet the performance requirements are outlined. If a defect is detected, it is important to understand the cause of the problem so that it can be avoided on future shafts and also so that the extent of the defect can be defined for evaluation purposes. Not all defects represent deficiencies that must be repaired. When repairs are required, there are a number of effective measures that can be employed as outlined in this chapter. The most effective repair technique depends upon the location and type of deficiency. A good quality control plan should be agreed upon and implemented before starting the remediation process. This can include effective use of core holes for the dual purpose of investigation and repair, pre-repair and post repair CSL or 3DT thermography, and location of post repair core holes and tests, if needed.

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CHAPTER 18
COST ESTIMATION

18.1 GENERAL

Estimating the cost of drilled shaft foundations is Step 15 in the overall design and construction process outlined in Chapter 8 (see Figure 8-1). During the planning phase of a project, preliminary estimates made by the design engineer are used to compare the costs of drilled shafts with other technically feasible foundation systems. These estimates may be one of the factors in selecting the foundation type. During final design, the engineer makes a cost estimate that will be used to estimate total project costs prior to bid. The *actual* cost of drilled shafts is established in the open marketplace through the process of competitive bidding. Unless there is a clear advantage to the use drilled shafts considering cost, environmental factors, site constraints, etc., the best way to obtain the lowest overall cost of a foundation for a highway construction project is to design competing foundation systems and to request bids on each of the systems. The additional engineering effort to develop alternative foundation designs is often more than offset by the savings in cost that will result.

This chapter briefly addresses the factors that impact costs of drilled shaft foundations and gives some general guidance on estimating costs.

18.2 FACTORS INFLUENCING COST

The cost of constructing drilled shafts will vary widely with geographic location and with the passage of time. The cost will also be largely affected by the quality and detail of the subsurface data available to the bidders and by the assumptions that the contractor makes regarding the construction method expected to be used. These and other factors are listed and discussed briefly in the following paragraphs:

- Subsurface and Site Conditions: This factor probably has the largest influence on the cost of construction. The difficulty of drilling will vary widely; for example, rock will be relatively soft in some locations and extremely hard at other places. Other site conditions will also have a big influence on cost, with the most important factors being: trafficability, nearby structures, traffic control, underground utilities, overhead lines, overhead bridge decks, trees, contours, and cut-off elevation of drilled shafts in relation to the ground surface. Difficult site access and difficult time restrictions such as night work and lane closures can have a large impact on cost. For example, costs of providing and operating low-headroom drill rigs are approximately three times the cost of rigs that operate in the open. Low headroom impacts time and cost in multiple ways, including the need to field splice multiple sections of casing and the reinforcement cage. Also, when low-headroom drilling is necessary, permanent casing is the norm because there is insufficient room or time for casing extraction.

- Geometry of Drilled Shaft: The cost per unit volume or length of drilling at a shallow depth will be less than that for a greater depth. In drilling soil, the unit cost of drilling per unit volume is less as the diameter of the hole increases until the hole becomes so large (typically larger than about 8-ft diameter) that readily available equipment cannot accomplish the drilling. In drilling rock, it is difficult to state the influence of diameter; however, holes with a larger diameter become increasingly more difficult to drill in hard rock.

- Number of drilled shafts on the project: Typically, projects with a larger number of drilled shafts will results in a lower unit costs since mobilization, planning, management, and testing costs are spread over a larger number of foundations.

- Specifications, including inspection procedures: Some specifications are written in such a manner as to make a large impact on cost; for example, if permanent casing is required where the job could be constructed as well or better without the use of the permanent casing. There are occasions when it is known that a "tough" (interpreted as "unreasonable") inspector will be on the job. Some contractors raise their prices to adjust to such a situation. Whether excavation is considered to be classified or unclassified can impact bid costs. Requirements for nondestructive testing and evaluation (NDT/NDE) will also increase bid costs, in particular if the contract calls for the contractor to be penalized for negative test results. NDT/NDE is covered in Chapter 16. As a general rule, specifications that dictate the use of a particular construction method (a "method spec") will drive up pricing. Specification issues are further discussed in Chapter 14, and a Guide Specification is presented in Appendix D.

- Obstructions: The issue of obstructions and how they are addressed can result in significant cost differences in large diameter drilling. Many owners pay the contractor for removal of obstructions, although some pay only for man-made obstructions. The no-obstruction projects receive large contingencies in their bids. The man-made-obstruction-only projects lead to medium sized contingencies, and the projects that pay all obstructions by cost reimbursement result in low contingencies.

- Expected weather conditions: Weather is an important factor regarding cost of construction. For example, harsh winter weather conditions can be expected to influence worker productivity. Contractors who work in these conditions are very much aware of the impact and will incorporate this into their bid prices.

- Work time restrictions Projects, particularly in urban areas, may have restrictions as to when work may be performed. Such restrictions decrease productivity and increase contractors' costs.

- Location of work as related to travel and living costs of crew: The cost of some projects is significantly greater than others because of location. Travel time and living costs can vary widely from place to place. Examples of locations with high travel and living costs are Alaska and Hawaii.

- Time allowed for the construction and penalty clauses: Some jobs are laid out on a very tight construction schedule and with a significant penalty if the work is not done on time. Drilled shafts can usually be constructed relatively rapidly, but the time for construction must be reasonable in order to control the cost.

- Work rules: The number of workers that are required for constructing a drilled shaft can vary from place to place. For example, in only some places is an oiler required. Also, restrictions on what a particular worker can do will vary. In some locations a certified welder is required for any welding that is needed, but in other locations the general workers on the job can perform tack welding. Costs will differ depending upon whether labor is union or non-union, and on the local prevailing wage requirements.

- Governmental regulations: The influence of governmental regulations on the cost of construction has been increasing in recent years as the sensitivity to environmental effects has increased. Restrictions concerning the pollution of the air and water have become more severe, and more attention is being placed on noise pollution. Also, regulations concerning job safety have become more stringent. Governmental regulations can vary from place to place in the United States.

- Availability of optimum equipment: There will normally be a number of pieces of equipment that will best fit the construction to be done. The availability of a wide variety of drilling equipment will vary with project location and the size of the drilled shafts.

- Availability and cost of concrete: In some markets, when there is high demand for ready-mix concrete, suppliers may assign top priority to customers purchasing larger quantities than drilled shaft contractors. This can make it difficult to schedule concrete pours to fit the scheduling needs of drilled shaft construction, and may cause volatility in concrete prices.

- Experience and ingenuity of contractor: Many experienced contractors have developed techniques that significantly reduce the cost of construction. Inexperienced contractors, on the other hand, may submit a low bid because of misjudgment of the difficulty of a particular job.

- Cost of materials: A sharp rise in the cost of materials, such as steel prices, can significantly affect the cost of drilled shafts. Where material costs may fluctuate with time, a contingency may be included in contractor bids.

- Economic conditions and amount of construction activity: The cost of construction will vary depending on the availability of work. The principle of supply-and-demand works strongly in the drilled shaft industry.

- Insurance and bonding: The cost of these items may impact the overall cost of construction,

- Cost of money and terms for payment: Interest rates have an impact on construction costs, and the schedule of payment to the contractor is an important factor.

- Terms of the contract: The cost of the construction will increase if the contractor is required to assume all risks, including the possibility that the actual site conditions are not the same as shown in the contract documents.

- General contractor's fees: Drilled shaft contractors are usually subcontractors to general contractors, who actually submit the bids for the project, so bid costs include the general contractor's fee for management, which can vary considerably among general contractors. A related factor is whether the general contractor is responsible for supplying and placing drilled shaft concrete. In some parts of the U.S. this is common practice; however, most drilled shaft contractors would prefer to have sole responsibility for the drilled shaft concrete in order to exert the appropriate quality control, an area in which they have more specialized knowledge and experience than many general contractors. A similar statement can be made regarding the supply and fabrication of rebar cages for drilled shafts. For large-diameter cages in particular, drilled shaft contractors are most familiar with the technical details of proper fabrication that will enable large cages to be lifted and placed safely without problems.

To provide a picture of the relative impact of various factors on drilled shaft costs, a survey of drilled shaft contractors was made in 1997, with details reported in a previous version of this manual (O'Neill and Reese, 1999). Participants were asked to submit a bid price per linear foot of drilled shaft to construct 50 drilled shafts in each of three scenarios. "Construction" of a drilled shaft was defined to mean drilling, tying and placing the steel, and placing the concrete. While the bid prices are no longer valid, some of the conclusions based on the survey are still meaningful and provide insight on the relative influence of several important factors:

- construction of sockets in hard rock is about three times as expensive, on average, as constructing a shaft through dry overburden above rock.

- when the overburden is alluvium that is wet and filled with boulders and the socket is in a soft geomaterial, overburden construction is about twice as expensive per unit of length as construction of the soft socket.
- drilling slurry or full-length casing is about one-third more expensive than constructing through dry, stable overburden
- The most uncertain cost is associated with excavating in hard rock; this likely reflects the different experiences of contractors with excavating the specific geologic formations in their respective market areas.

18.3 COMMENTARY

Considering the number of variables affecting the cost of drilled shafts, there is no single formula or method that will provide highly accurate costs for a specific project prior to bidding. Contractors typically estimate their costs based on consideration of time and materials. However, as described in the next section, local cost information based on contracts awarded (historical information) can be used as a guide to make reasonable preliminary estimates for comparing drilled shafts to other foundation types. For final estimates prior to bid, historical price information combined with experience and knowledge of constructability and its influence on costs are the basis for making good estimates.

Whether the cost of foundations is relatively small or represents a large percentage of the total cost of the project, the prudent action is to select a contractor who has the necessary equipment, experienced personnel, and a record of high-quality construction so that the foundation can be built in rapid order and with good quality. In order to ensure this situation, some agencies have a requirement that the drilled shaft contractor be prequalified, which should aid in obtaining work of high quality.

18.4 PUBLICALLY AVAILABLE PRICE DATA

Most state transportation agencies maintain a data base of submitted bid prices ('bid tabs') which can be accessed via a website. Historical price information is a valuable resource to engineers (and contractors) because it provides a snapshot of actual costs of drilled shafts in a particular state and, in some cases, in a particular geographic region within the state. Table **18-1** is an example of a low-bid summary table that was obtained from a website maintained by the Texas Department of Transportation (TxDOT). Texas provides a good example because TxDOT uses a relatively large number of drilled shafts and the market is competitive, with numerous qualified drilled shaft contractors bidding on TxDOT work.

Table 18-1 is a summary for the entire state. The geology across Texas varies considerably and, therefore, the subsurface conditions among jobs represented in Table 18-1 undoubtedly also varied considerably. Also, the lengths of drilled shafts, which are not tabulated, most likely varied over a wide range. It is important to recognize that the information in this type of table cannot be used to pinpoint costs for any particular drilled shaft project because many of the factors that influence the bids for a particular job (*e.g.*, subsurface conditions, length of shafts, access, method for ground and groundwater control, etc.) are not identified. However, the data provide a general picture of costs relative to the diameter of the drilled shaft and, by comparing the latest bids with the 12-month moving average, provide a general idea of the pricing trends in Texas. These data should not be considered reliable for other geographic regions of the country, considering geologic conditions, as well as other factors, may differ considerably.

TABLE 18-1 LOW-BID TABLE FOR DRILLED SHAFTS, TEXAS DOT, STATEWIDE

Description	Month ending 5/31/2018		12-Month Moving Average		
	Quantity[1]	Average Bid[2]	Quantity[1]	Average Bid[2]	No. of Projects
DRILL SHAFT (30 IN)	10,106	$104.22	69,726	$109.05	32
DRILL SHAFT (36 IN)	40,978	$153.18	213,823	$156.97	89
DRILL SHAFT (42 IN)	16,874	$159.43	64,042	$194.97	31
DRILL SHAFT (48 IN)	18,283	$254.71	49,076	$276.47	40
DRILL SHAFT (54 IN)	5,433	$322.00	20,604	$285.77	18
DRILL SHAFT (60 IN)	7,659	$359.00	32,825	$371.33	10
DRILL SHAFT (66 IN)	900	$451.00	2,504	$474.00	5
DRILL SHAFT (72 IN)	0	-	7,743	$729.00	6
DRILL SHAFT (78 IN)	0	-	783	$658.56	2
DRILL SHAFT (84 IN)	1,846	$678.00	10,378	$836.68	9
DRILL SHAFT (SIGN MTS) (30 IN)	0	-	1,434	$202.42	4
DRILL SHAFT (SIGN MTS) (36 IN)	0	-	3,977	$203.74	9
DRILL SHAFT (SIGN MTS) (42 IN)	1,110	$389.00	4,295	$281.58	10
DRILL SHAFT (SIGN MTS) (48 IN)	425	$478.87	2,387	$442.61	15
DRILL SHAFT (SIGN MTS) (54 IN)	0	-	1,.250	$439.18	6
DRILL SHAFT (HIGH MAST POLE) (54 IN)	0	-	90	$650.00	1
DRILL SHAFT (HIGH MAST POLE) (60 IN)	234	$378.85	6,213	$496.00	16
DRILL SHAFT (HIGH MAST POLE) (66 IN)	0	-	1,186	$584.20	4
DRILL SHAFT (RDWY ILL POLE (30 IN)	2,210	$178.69	12,018	$192.05	75
DRILL SHAFT (TRF SIG POLE) (30 IN)	67	$241.79	1,799	$247.16	45
DRILL SHAFT (TRF SIG POLE) (36 IN)	1,478	$275.26	11,170	$269.14	115
DRILL SHAFT (TRF SIG POLE) (42 IN)	17	$350.00	188	$335.80	7
DRILL SHAFT (TRF SIG POLE) (48 IN)	974	$381.68	4,770	$372.02	55
DRILL SHAFT (108 IN)	2,110	$1,115.00	5,950	$1,078.84	5
DRILL SHAFT (144 IN)	0	-	1,324	$1,570.47	2
DRILL SHAFT (96 IN)	10,736	$899.41	19,108	$831.99	5
DRILL SHAFT (120 IN)	1,334	$1,390.00	4,156	$1,394.95	4
DRILL SHAFT (TRF SIG POLE) (30 IN) (ROCK)	0	-	61	$271.69	3
DRILL SHAFT (TRF SIG POLE) (36 IN) (ROCK)	0	-	178	$273.53	4
DRILL SHAFT (SIGN MTS) (60 IN)	0	-	422	$671.56	2
DRILL SHAFT (RDWY ILL POLE (24 IN)	152	$283.00	797	$158.43	4
DRILL SHAFT (132 IN)	0	-	344	$1,560.57	1
DRILL SHAFT (TRF SIG POLE) (42 IN) (ROCK)	0	-	17	$330.00	1
DRILL SHAFT (TRF SIG POLE) (48 IN) (ROCK)	0	-	88	$425.00	1

(Source: TXDOT)

1 All quantities are in linear feet

2 All bid prices are in dollars per linear foot

Table 18-2 shows drilled shaft bid prices submitted by six different contractors for a single highway project in Galveston, TX that included a wide range of drilled shaft sizes. The bidders are numbered 1 through 6, corresponding to lowest to highest bid submitted for the entire project. The third column is the Engineer's Estimate for comparison to the bid prices. Note the wide range in contractor bid prices for some of the shaft sizes, and also note that the winning bid (Bidder 1) did not necessarily have the lowest price for drilled shafts. Drilled shafts represented approximately 4 percent of the cost incorporated into the winning bid. This would be a low percentage of total cost for a bridge, but the project in this case involved construction of 5 miles of interstate highway and construction costs of roadway and other features were more significant.

TABLE 18-2 BID TABS FOR DRILLED SHAFTS ON A SINGLE PROJECT, GALVESTON

Description	Quantity[1]	Estimated Unit Cost[2]	Bidder	Unit Bid Price
DRILL SHAFT (30 IN)	3,310	$107.63	1	$125.00
			2	$107.00
			3	$101.62
			4	$150.00
			5	$110.00
			6	$130.00
DRILL SHAFT (36 IN)	18,233	$146.57	1	$150.00
			2	$155.00
			3	$151.47
			4	$180.00
			5	$150.00
			6	$190.00
DRILL SHAFT (42 IN)	11,577	$215.00	1	$200.00
			2	$200.00
			3	$202.12
			4	$250.00
			5	$220.00
			6	$240.00
DRILL SHAFT (48 IN)	22	$230.00	1	$250.00
			2	$560.00
			3	$550.00
			4	$375.00
			5	$225.00
			6	$210.00
DRILL SHAFT (60 IN)	864	$283.30	1	$400.00
			2	$445.00
			3	$407.00
			4	$500.00
			5	$400.00
			6	$420.00
DRILL SHAFT (66 IN)	864	$395.58	1	$500.00
			2	$528.00
			3	$488.57
			4	$595.00
			5	$470.00
			6	$550.00
DRILL SHAFT (72 IN)	1,204	$475.001	1	$600.00
			2	$655.00
			3	$596.39
			4	$710.00
			5	$580.00
			6	$625.00
DRILL SHAFT (78 IN)	156	$595.40	1	$700.00
			2	$745.00
			3	$705.32
			4	$835.00
			5	$690.00
			6	$725.00
DRILL SHAFT (84 IN)	972	$650.00	1	$800.00
			2	$880.00
			3	$798.03
			4	$900.00
			5	$780.00
			6	$815.00

(Source: TXDOT)

1 All quantities are in linear feet

2 All prices are in dollars per linear foot

A second example of state-wide cost information for drilled shafts is presented in Table 18-3 for California, which also is a competitive market and a state that utilizes drilled shafts extensively. In addition to providing a comparison to costs in Texas, Table 18-3 shows how different subsurface conditions affect cost. For example, the average unit cost of 24-inch diameter rock sockets ($390 per ft) based on two projects, was approximately 1.75 times the cost of 24-inch diameter shafts in soil ($223 per ft) based on six projects. A single project with 72-inch diameter shafts shows a higher unit cost ($1,518 per ft) than the single-project unit cost of 96-inch diameter drilled shafts ($1,462 per ft), suggesting that factors other than shaft size controlled the cost on these particular projects. Note that factors such as the amount of reinforcing steel, which can be driven by seismic considerations especially in California, are not identified in bid tabs but may have a cost impact.

In using historical price data, it is also important to understand what items of work are included in the bid. Some states have separate bid items for drilled shaft excavation and drilled shaft concrete. Permanent steel casing may also be a separate bid item. Related cost items, such as mobilization and load testing, typically would be separate bid items. In addition, other non-shaft work items, such as site preparation, traffic control, concrete for pile caps, and others, are not included in the drilled shaft bid items, but may impact the total cost of drilled shaft foundation construction.

TABLE 18-3 AVERAGE LOW-BID PRICES FOR DRILLED SHAFTS, 2017

Description	Average Values, 2017		
	Quantity[1]	Average Unit Price[2]	No. of Projects
16” Cast-in-Drilled-Hole Concrete Piling (Barrier)	6,694	$72.62	3
16” Cast-in-Drilled-Hole Concrete Piling (Sound Wall)	2,100	$117.00	1
16” Cast-in-Drilled-Hole Concrete Piling	466	$163.52	2
24” Cast-in-Drilled-Hole Concrete Piling	2,719	$222.89	6
24” Cast-in-Drilled-Hole Concrete Piling (Rock Socket)	2,237	$389.64	2
30” Cast-in-Drilled-Hole Concrete Piling (Sign Foundation)	32	$700.00	1
30” Cast-in-Drilled-Hole Concrete Piling	2,982	$119.29	1
36” Cast-in-Drilled-Hole Concrete Piling (Sign Foundation)	20	$540.75	1
36” Cast-in-Drilled-Hole Concrete Piling	1,620	$511.65	3
42” Cast-in-Drilled-Hole Concrete Piling	188	$1,144.00	1
48” Cast-in-Drilled-Hole Concrete Piling (Sign Foundation)	130	$580.00	1
48” Cast-in-Drilled-Hole Concrete Piling	264	$900.00	1
54” Cast-in-Drilled-Hole Concrete Piling (Sign Foundation)	46	$815.00	2
60” Cast-in-Drilled-Hole Concrete Piling (Sign Foundation)	1,726	$999.04	17
72” Cast-in-Drilled-Hole Concrete Piling	175	$1,518.30	1
96” Cast-in-Drilled-Hole Concrete Piling	480	$1,462.00	1
108” Cast-in-Drilled-Hole Concrete Piling	234	$1,619.29	1

(Source: CALTRANS)

1 All quantities are in linear feet

2 All bid prices are in dollars per linear foot

18.5 CONTRACTORS' COST COMPUTATION

The calculation of a bid for drilled shafts is made up of a number of components. Some costs are variable and are a function of the quantities of various items such as concrete, steel, excavation, etc. Other costs, such as the cost of engineering, bonding, and moving of equipment to and from the jobsite, are unique to each project, but are viewed as fixed because they do not vary with the quantity of work. Still other costs are viewed as overhead, which represents those costs which are ongoing to run the company regardless of the amount of work undertaken.

In the preparation of a quotation for drilled shafts, the Drilled Shaft Contractor will first examine the contract documents including geotechnical reports, boring logs, drawings, and specifications. Site conditions are then appraised from a site visit. The next step is to establish a tentative work plan that defines the methods to be used to complete the work, equipment to be used, and manpower requirements. Once a work plan is established, a quantity takeoff of materials to be used on the project can be calculated. This would include concrete, steel, NDT testing, slurry, and any other required materials.

Next, a rough schedule must be determined. A careful review of specifications must precede this step as often there are contract requirements which affect schedule. Examples are restrictions on drilling adjacent to recently poured shafts, and time waiting for NDT testing and subsequent approval by the resident engineer.

18.5.1 Variable Costs

Labor: With the rough schedule in hand, the contractor can estimate labor costs including fringe benefits, payroll taxes, etc. for the project. The estimate must include allowances for anticipated overtime and may also include the cost of unproductive time because of inclement weather or other causes (see Contingencies).

Materials: The material takeoff is now priced by contacting suppliers. Material quantities include any anticipated overruns or waste materials such as the added concrete to overpour the shafts or that which may be anticipated by overbreak in the drilling process. Reinforcing steel and NDT testing are usually procured through subcontractors or suppliers if these items are to be included in the drilled shaft contractor's scope of work. NDT testing tubes are sometimes tied in the cages by the contractor's own forces, while other times the reinforcing steel supplier will provide these items. Either way, the estimator must appreciate where this item is included in the estimate.

Rented Equipment: Any rented equipment is priced based on quotations from suppliers and the cost calculated using the schedule developed at the start of the estimate.

Contractor-Owned Equipment: Several different approaches are used by contractors to incorporate the cost of their equipment in bids. Some contractors will develop very sophisticated data bases which permit them to identify all the costs of owning and running equipment. These include:

- Capital cost of purchase
- Cost of financing
- Cost of insurance
- Depreciation
- Major repairs
- Allowance for wear parts (cable, wear pads, filters etc.)
- FOG (fuel, oil, grease)
- Others

An estimate is made at the start of each year as to the number of operating hours the contractor anticipates for each piece of equipment. It is then possible to create an hourly equipment charge for each piece of equipment for use in estimating. These charges are then collected into an Equipment Costing account which, hopefully, will balance at the end of the year.

Other drilled shaft contractors may use published equipment costs such as Dataquest, Blue Book, or State Force-Account rates.

Still others may include only the fuel, oil and grease (FOG) required to operate a piece of equipment as a variable cost item in the estimate and place all other costs of equipment in the Overhead category. Whichever method is used, the drilled shaft contractor will then calculate equipment costs as a function of equipment hours, in the same way labor costs are estimated.

18.5.2 Jobsite Fixed Costs

A separate estimate is prepared which calculates the cost of:

- Moves to and from the site
- Moving rigs between foundation locations (this may be a significant cost for offshore work)
- Costs of establishing site trailers, etc.
- Bonding
- Engineering
- Supervision (some contractors will carry this in their variable costs)
- Project Management (some contractors will carry this in their Overhead)
- Other fixed costs

18.5.3 Overhead

The cost of running the corporation must be written off over the year's work. Some drilled shaft contractors may cover overhead costs as a percentage of total job costs, others as a daily charge against major equipment. Still others may cover it in their markup. Overhead costs may include:

- Management salaries and benefits
- Administration such as Payroll, Accounts Receivable (AR), and Accounts Payable (AP)
- Telephones and Information Technology (IT)
- Over-runs or under-runs in the equipment costing account.
- Professional fees (legal, accounting, etc.)
- Insurance cost
- Research and Development (R&D)
- Office rent and expenses
- Yard expenses (some contractors charge these back to each job)
- Equipment costs not charged to a project (see section on equipment costing)
- Others

18.5.4 Contingencies

Some contractors may make a separate estimate of contingent items while others may cover it in their markup. Contingencies include allowances for:

- Inclement weather
- Labor disruption
- Repair of anomalies
- Unit prices for materials
- Delays due to equipment repairs and maintenance
- Schedule disruption caused by others
- Potential differing site condition (DSC) claims which may be difficult to collect
- Potential Liquidated Damages
- Others

18.5.5 Markup

Some drilled shaft contractors may total all of the above costs and add a percentage markup to arrive at final pricing. Others may mark up different types of costs with different percentages based on the uncertainty of each item. For instance, for contractors who use this system, labor is traditionally much more volatile than subcontractor costs and so the markups reflect the uncertainty inherent.

It is not possible to quote reasonable markups without a very clear understanding of where a contractor's cost are being carried and if the markup includes some of those costs or whether all costs are separately calculated and the markup is only gross profit.

Markup will also vary based on market forces, the amount of backlog a contractor may or may not have, and the contractor's comfort level with the type of work, location of the work, and the track record of the client.

18.5.6 Unit Prices

Once the total bid price of the project is calculated, costs must be assigned to individual bid items which typically are designated by the owner. Some of the bid items will be in terms of unit prices, while others will be fixed or lump sum. The drilled shaft contractor must then make an appraisal of how volatile the job quantities may be. Are shafts likely to be longer or shorter than the engineer's estimate? Is more or less casing likely to be needed? The contractor will then make a decision as to how much of the total cost will be distributed across the unit priced items (those subject to fluctuation) and how much will be reserved against lump sum items such as Mobilization, Engineering, etc. This can be a risky part of the estimating process and even a good estimate can result in significant losses if costs are spread in a manner which does not end up favorably when the final quantities are known.

18.5.7 Other Considerations

When comparing unit prices for an item of work, an engineer should have a very clear understanding of what is included and what is not included. Examples of items which could be included in a drilled shaft contract, but which may be supplied by the general contractor or other parties, are concrete, reinforcing steel, permanent casing, load testing, NDT testing, and others.

18.6 TYPICAL COST ESTIMATING TEMPLATE

An example template for estimating costs of bridges is presented in Figure 18-1. This template, used by the Florida DOT, is based on a 3-step procedure in which costs of various components of the substructure, including drilled shafts, and superstructure are first estimated using historical cost data from bid tabs and other sources. In Step 2, the total cost estimate is adjusted for factors such as rural versus urban construction, construction over water, or phased construction. In step 3, a 'reality check' is performed by comparing overall cost per square foot with historic costs for bridge types similar to the bridge being designed. For brevity, some of the steps involving superstructure elements are omitted. Note that some costs included in the example template are outdated, but the template serves to illustrate how a state agency may make cost estimates for purposes of budgeting and for comparison to actual bid prices.

9.2 BDR Bridge Cost Estimating

The applicability of this three-step process is explained in the general section. The process stated below is developed for estimating the bridge cost after the completion of the preliminary design, which includes member selection, member size and member reinforcing. This process will develop costs for the bridge superstructure and substructure from beginning to end bridge. Costs for all other items including but not limited to the following are excluded from the costs provided in this chapter: mobilization, operation costs for existing bridge(s); removal of existing bridge or bridge fenders; lighting; walls; deck drainage systems; embankment; fenders; approach slabs; maintenance of traffic; load tests; bank stabilization.

Step One:
Utilizing the cost provided herein, develop the cost estimate for each bridge type under consideration.

9.2.1 Substructure

A. Prestressed Concrete Piling; cost per linear foot (furnished and installed)

Size of Piling	Driven Plumb or 1" Batter	Driven Battered
18-inch	$38	$47
24-inch	$53	$67
30-inch	$63	$80

When heavy mild steel reinforcing is used in the pile head, add $250.
When silica fume is used, add $6 per LF to the piling cost.

B. Steel Piling; cost per linear foot (furnished and installed)

Steel Piling	Cost
14" x 73 H Section	$35
14" x 89 H Section	$38
20" Pipe Pile	$84
24" Pipe Pile	$90
30" Pipe Pile	$152

C. Drilled Shaft; total in-place cost per LF

Drilled Shaft	3 ft	4 ft	5 ft	6 ft	7 ft	8 ft	9 ft
On land with casing salvaged.	$239	$277	$340	$441	$542	-	-
In water with casing salvaged.	$277	$302	$353	$479	$605	$806	-
In water with permanent casing.	$428	$466	$554	$643	$781	$970	$1184

D. Sheet Piling Walls

Sheet Piling Item	Wall	Cost
Prestressed concrete; cost per linear foot	10" x 30"	$71
	12" x 30"	$86
Steel; cost per square foot	Permanent Cantilever Wall	$20
	Temporary Cantilever Wall	$6

E. Cofferdam Footing (cofferdam and seal concrete)
Prorate the cost provided herein based on area and depth of water. A cofferdam footing having the following attributes will cost $328,000.

Area; 63 ft x 37.25 ft. Depth of seal; 5 ft. Depth of water over the footing; 16 ft.

F. Substructure Concrete; cost per cubic yard.

Substructure Item	Cost
Concrete:	$550
Mass concrete:	$314
Seal concrete:	$344
Shell fill:	$5

For calcium nitrite, add $32 per cubic yard. (@4.5 gal per cubic yard).
For silica fume, add $25 per cubic yard. (@60 lbs. per cubic yard.)

G. Reinforcing Steel; cost per pound; $0.46

Step Two:

After developing the total cost estimate utilizing the unit cost, modify the cost to account for site condition variables. If appropriate, the cost will be modified by the following variables:

1. For rural construction decrease construction cost by 6 percent.
2. For urban construction (Broward, Dade, Duval, Hillsborough, Orange, Palm Beach and Pinellas counties), increase construction cost by 6 percent.
3. For construction over water increase construction cost by 3 percent.
4. For phased construction (over traffic or construction requiring multiple phases to complete the entire cross section of the bridge), add a 20 percent premium to the affected units of the structure.

Step Three:

The final step is a comparison of the cost estimate with historic bridge cost per square foot data. These total cost numbers are calculated exclusively for the bridge cost as defined in the General Section of this chapter. Price computed by Steps 1 and 2 should be generally within the range of cost of as supplied herein. If the cost falls outside the provided range, good justification must be provided.

Bridge Superstructure Type		**Total Cost per Square Foot**
Reinforced Concrete Flat Slab; Simple Span		$50-65*
Reinforced Concrete Flat Slab; Continuous Span		$60-80*
Steel Deck/Girder; Simple Span		$62-75*
Steel Deck/Girder; Continuous Span		$70-90*
Prestressed Concrete Deck/Girder; Simple Span		$50-70*
Prestressed Concrete Deck/Girder; Continuous Span		$65-110*
Post-tensioned, cast-in-place Concrete Box Girder		
Cast on scaffolding; span length <= 240 ft		$75-110
Steel Box Deck/Girder		
Span range from 150 ft to 280 ft.		$76-120
For curvature add a 15 percent premium		
Segmental Concrete Box Girders		
Span range from 150 ft to 280 ft		$80-110
Movable Bridges; bascule spans & piers		$900-1500
Demolition of existing bridges	Typical	$9-15
	Bascule spans & piers	$63

*Increase the cost by 20 percent for phased construction.

Figure 18-1 Example Cost Estimating Template (Source: Florida DOT)

18.7 SUMMARY

The factors that determine costs of drilled shaft construction are identified and discussed. Site conditions, including both subsurface and surface features, often are the most significant factors influencing the overall costs. The use of cost data from recent projects is shown to be a valuable tool for making engineering cost estimates; however, the cost of drilled shafts can be highly project-specific and there is no single reliable method of predicting actual costs. The general process used by contractors for estimating drilled shaft costs is outlined and discussed. Having a general idea of how contractors prepare bid costs can be of benefit to engineers in making estimates of costs for planning purposes and for comparing various foundation options.

CHAPTER 19
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APPENDIX A

DESIGN EXAMPLE: DRILLED SHAFTS FOR REPLACEMENT BRIDGE

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APPENDIX A
DESIGN EXAMPLE: DRILLED SHAFTS FOR REPLACEMENT BRIDGE

An example is used to illustrate design of drilled shafts as foundations for a proposed replacement bridge. Conditions are based on an actual project. The owner is a state transportation agency and the site is located in the Atlantic Coastal Plain physiographic province. Design of the drilled shaft foundations is used to illustrate the LRFD design procedures presented in this manual. The following aspects of drilled shaft design and corresponding chapters in this manual are incorporated into the example:

Chapter 2: Site Characterization
Chapter 8: Drilled Shaft Design Process
Chapter 9: Design for Lateral Loading
Chapter 10: Geotechnical Design for Axial Loading
Chapter 12: Structural Design

A.1 GENERAL

The example follows the process outlined in Figure 8-1 "Drilled Shaft Design and Construction Process". It is assumed that the general project requirements and constraints have been established and preliminary studies on various bridge and foundation options are completed (Steps 1 through 6) and that a decision has been made to proceed with further evaluation of drilled shafts (Step 7). A minor deviation is made from the step-by-step procedure, in that Step 11 (design for axial load) is presented before Step 10 (design for lateral load); because in this case the axial load considerations control the drilled shaft final dimensions.

The proposed replacement bridge is a 6-span structure, 1,230 ft in length and 57 ft wide, supported on two abutments and five piers. The superstructure consists of AASHTO Modified Type VI spliced post-tensioned girders. Each abutment is supported on a group of driven piles with the piers supported on drilled shafts. Span lengths range from 175 to 220 ft and the trial design calls for three, 8-ft diameter drilled shafts per pier. Each drilled shaft supports an 8-ft diameter concrete pier column, and each set of three pier columns supports a cast-in-place concrete pier cap. The joint between each drilled shaft and pier column is located 1 ft above the mean high water level (MHW). For this example, drilled shaft design is focused on Pier No. 2. Figure A-1 shows a cross section of the bridge at Pier 2.

Step 8: Define Subsurface Profile for Analysis

A subsurface exploration program consisting of 32 borings was performed, in addition to 32 borings made in 1953 for an existing bridge at the project site. Geologic studies combined with the boring logs show that soils in the project area consist of Pleistocene lowland alluvial deposits overlying Tertiary marine sediments. In some areas of the site the alluvial deposits are covered by recent marsh soils which were deposited in tidal estuaries along the coastal areas and inland streams of the Atlantic coastal plain. The tidal marsh deposits consist of compressible organic silts and peat extending from mudline to depths of 10 to 30 ft. The underlying Pleistocene alluvial deposits are composed of layers of yellow-brown to gray-white quartz sand and gravel occurring in well defined alternating or intermixed layers. Silts and clay are encountered in lenses or pockets. Tertiary marine deposits underlie the Pleistocene sediments and consist of medium to fine silty sands, white to yellow in color, with localized gray to brown clay layers and lenses.

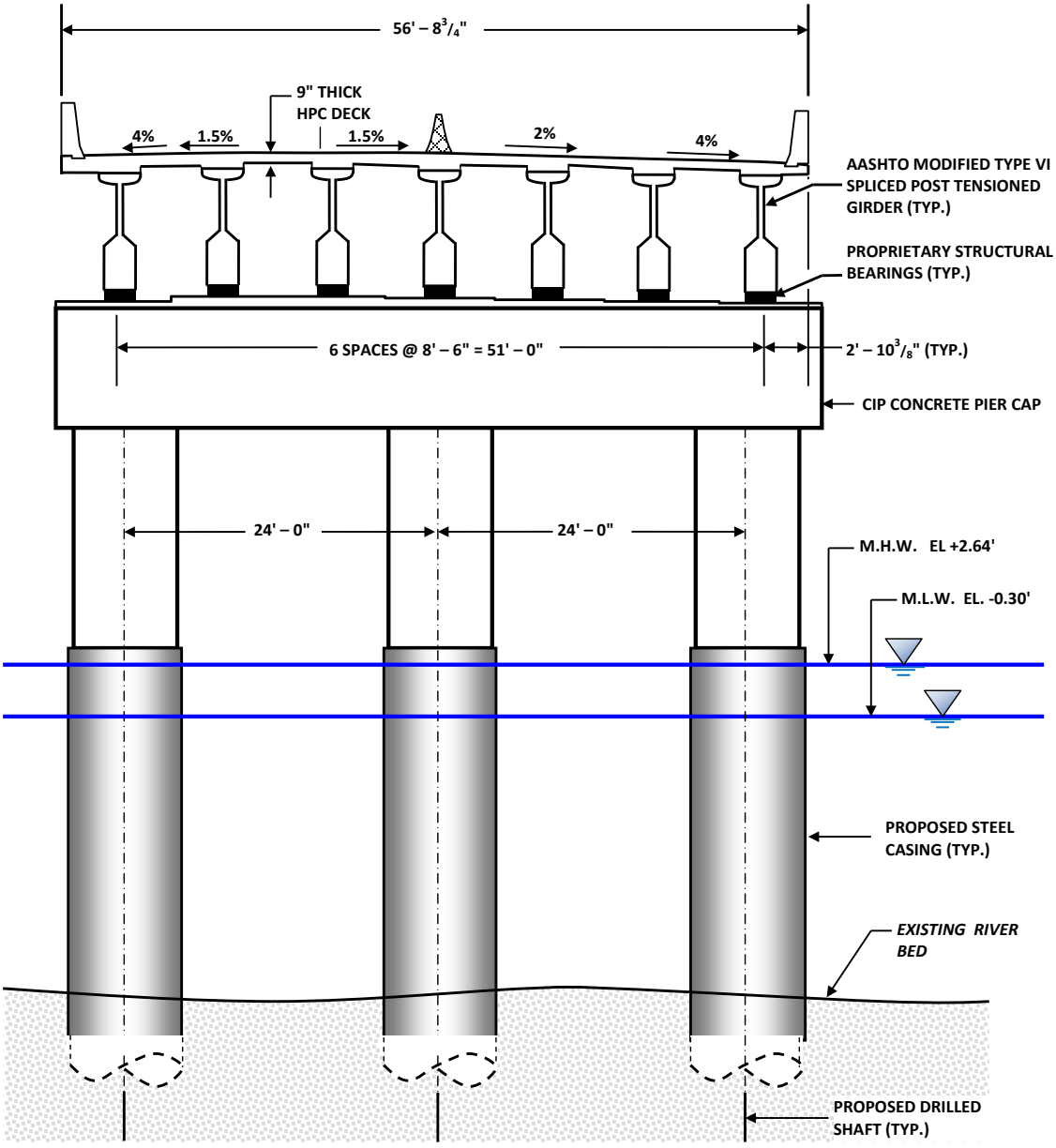


Figure A-1 Pier Section of Proposed Bridge

Figure A-2 shows a generalized subsurface profile along the alignment of the bridge. Soils are subdivided into general strata showing similar properties as described in the Legend of Figure A-2. At Pier No. 2, the reference boring is Boring W-2. Five of the general soil strata are present at Pier 2; these include strata R, C, D, E and F. Stratum R, river bed material consisting of loose sand and organic soils, is approximately 3.5 ft thick at Pier 2 and is neglected. The uppermost strata at Pier 2 is therefore Stratum C, which starts at elevation -29 ft. Field N-values from the Standard Penetration Test at Boring W-2 are also shown at 5 ft intervals and these values are used to obtain soil engineering properties for drilled shaft design.

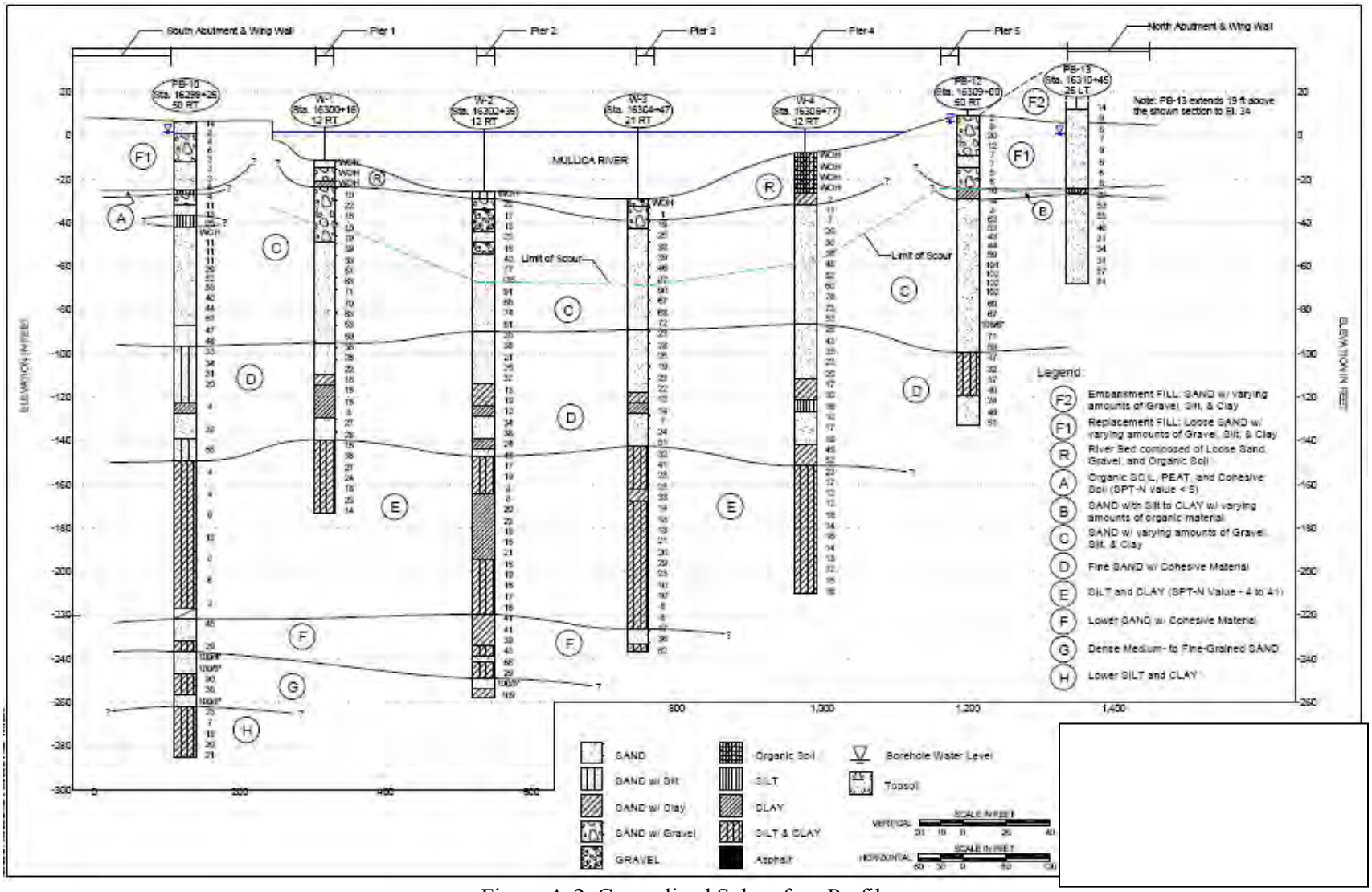


Figure A-2 Generalized Subsurface Profile

A.2 SCOUR AND SEISMIC CONSIDERATIONS

The profile in Figure A-2 shows the scour line, based on hydrologic studies in accordance with methods described in Section 10.5 of this manual. The scour limit shown is for the scour design flood and results in total scour of 38 ft at Pier 2. This scour depth results in removal of the upper portion of Stratum C at Pier 2, to an elevation of -67 ft. The resulting subsurface profile (*i.e.*, with full scour) is used to evaluate Strength I and Service I limit states in subsequent design steps.

The proposed bridge is evaluated for Extreme Event I limit state (earthquake). Although the bridge is determined to be in Seismic Performance Zone 1, the owner-agency designated the bridge as "essential" and required a site-specific response analysis. These analyses were carried out for selected locations along the approach piers and the main span units to account for the variations in subsurface soil conditions along the bridge alignment. The final design ground motions were determined based on the results of site specific response analyses and the following seismic design parameters were established:

Peak Ground Acceleration, 1,000-year event: A = 0.052g
Site Class: D (stiff soils)
Seismic Importance Category: Essential
Seismic Performance: Zone 1
Elastic Response Coefficient: C_{sm} = 0.075

Horizontal force effects due to earthquake (EQ) were determined using the elastic response coefficient (C_{sm}) and the equivalent weight of the superstructure, as described in Chapter 9 and in accordance with AASHTO LRFD Bridge Design Specifications (2017). Structural modeling of the bridge under the Extreme Event I load combination was used to determine the force demands transmitted to the drilled shaft foundations. In accordance with the owner specifications, Extreme Event I conditions did not include scour.

Within the limits of the new bridge piers, liquefaction analysis indicates that the existing fill and underlying sand deposits are generally medium dense to dense and not subject to liquefaction. Although small pockets of loose soils do exist, it was determined that liquefaction of the localized fill material will not adversely affect the drilled shaft foundations.

Step 9: Establish Limit States and Factored Drilled Shaft Load Demands

The following limit states are to be checked for the drilled shafts supporting the bridge piers:

Strength I (including effects of scour at the design flood):
- Geotechnical axial compression resistance of single shaft
- Lateral geotechnical resistance of single shaft
- Structural resistance of single shaft, including checks for axial, lateral, and flexural resistances

Extreme Event I: Load combination including earthquake:
- Geotechnical axial compression resistance of single shaft
- Lateral geotechnical resistance of single shaft
- Structural resistance of single shaft, including checks for axial, lateral, and flexural resistances

Service I (including effects of scour at the design flood):

Settlement (vertical deformation) of single shaft

Horizontal movement at the top of the foundation

Factored force demands were determined by structural modeling of the bridge using a commercially available finite element computer program. Initially, drilled shafts were modeled as 8.5-ft diameter equivalent fixed-end columns extending from the joint with the pier column (1 ft above MHW) to a point of fixity 15 ft below the mudline (10 ft below scour line for limit states that include scour).

The diameter of 8.5 ft accounts for permanent casing, which is used from 2 ft below MHW to a depth corresponding to the final scour elevation (-67 ft). Figure A-3 shows an example of the frame geometry for the structural model at Pier 2. An additional node was added to each drilled shaft element at the mudline elevation (elevation -29 ft). Force effects calculated at these mudline nodes were taken as the force effects applied to the tops of the drilled shafts in the geotechnical analysis. Additional nodes were also added at several intermediate depths below the mudline for modeling the lateral stiffness of the embedded shafts in subsequent iterations.

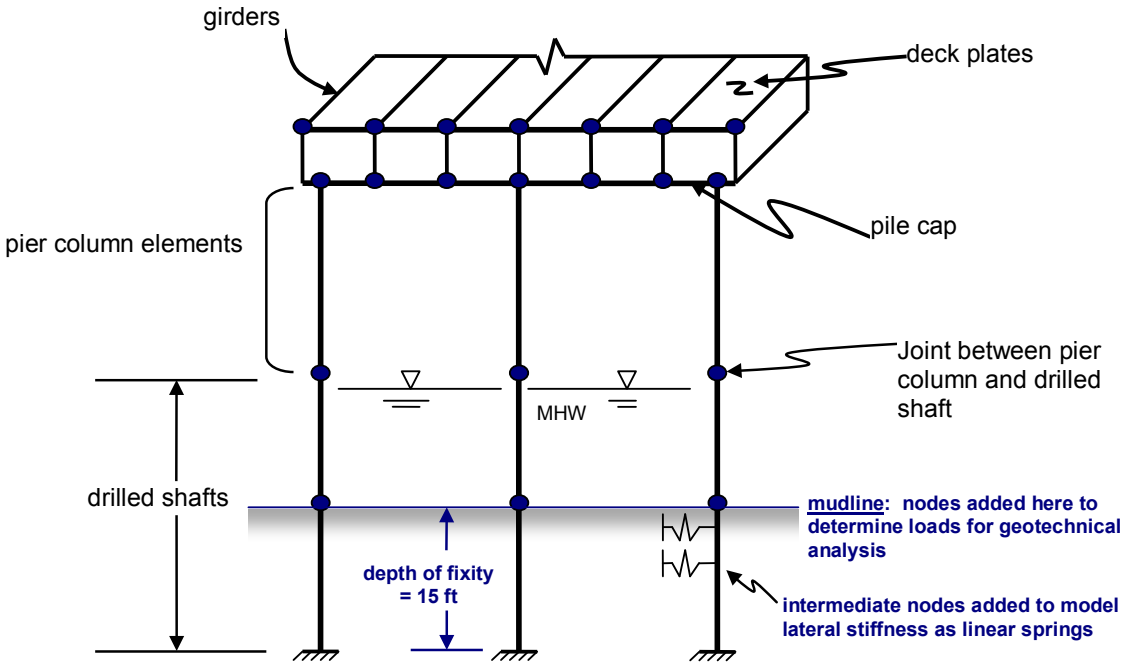


Figure A-3 Idealized Frame Model for Structural Analysis, Pier 2

The iterative procedure for establishing factored force effects on the drilled shafts can be summarized as follows:

1. For each applicable limit state, the structural model is analyzed under the factored loads; load combinations and load factors are those specified in Tables 8-2 and 8-3; drilled shaft lateral spring constants initially are assumed, in consultation with geotechnical specialists
2. Force effects calculated by the structural modeling program from Step 1 at the mudline nodes are resolved into axial, lateral, and moment components
3. The factored axial, lateral, and moment force effects from Step 2 are used as the applied loads for analyzing trial drilled shafts by the *p-y* method of analysis; *p-y* curves are established on the basis

of soil properties and the idealized subsurface profile; the top of each drilled shaft corresponds to mudline elevation (-29 ft)

4. Results of the *p-y* analysis are used to evaluate lateral stiffness versus lateral deflection along the length of the drilled shaft; these stiffness profiles are used to develop revised spring constants in subsequent structural modeling
5. The structural model is re-analyzed using the revised lateral spring constants established in Step 4; drilled shafts are modeled as beam-column elements; these elements are assumed to be pinned at their lower end, at a depth corresponding to zero lateral deflection from the *p-y* analysis
6. Steps 3, 4, and 5 are repeated iteratively until lateral deflections at mudline elevation calculated by the structural modeling program and *p-y* analyses show agreement to within $\pm$ 20 percent.
7. Factored force effects (foundation load demands) from structural modeling at the mudline nodes are used to conduct LRFD checks on all applicable limit states specified by AASHTO and summarized above.

Factored axial, lateral, and moment load demands at the top-of-shaft resulting from the analysis procedure outlined above are summarized in Table A-1 for the drilled shafts at Pier 2. In subsequent sections, these factored loads are used in limit state checks to show that the drilled shaft design satisfies the basic LRFD requirement, that the factored demands in Table A-1 may not exceed the summation of factored resistances.

TABLE A-1 SUMMARY OF DRILLED SHAFT FACTORED LOAD DEMANDS, PIER 2

Limit State	Max Axial Compression (kips)	Min Axial Compression (kips)	Lateral, Longitudinal (kips)	Lateral, Transverse (kips)	Moment about Longitudinal Axis (kip-ft)	Moment about Transverse Axis (kip-ft)
Strength I	3,389.47	2,673.42	36.98	9.63	1,686.87	221.68
Service I	2,835.40	1,836.42	54.65	34.59	2,647.17	317.24
Extreme Event I	2,823.04	2,810.26	144.52	290.48	7,776.42	3,212.32

Step 11: Establish Diameter and Depths for Axial Loads

Reference is made to the step-by-step procedure shown in Figure 10-2, "Flow Chart of Recommended Steps for Axial Load Design." Each step is carried out as follows.

Step 11.1 At each foundation location, divide the subsurface into a finite number of geomaterial layers; assign one of the following geomaterial types to each layer: cohesionless soil; cohesive soil**;** rock; IGM. Figure A-4 shows the idealized geomaterial layer profile used for foundation design at Pier 2, based on Boring W-2. Table A-2 presents the depth and elevation limits of each geomaterial, the general stratum to which each layer corresponds, and the geomaterial type. All of the geomaterials are categorized as either cohesionless soil or cohesive soil. For Strength I and Service I limit states, full scour is assumed, resulting in complete removal of Layer No. 1 and partial removal of Layer No. 2 to a depth of 38 ft (elevation - 67 ft.). For Extreme Event I, zero scour is assumed.

Step 11.2 Review the limit states to be satisfied and the corresponding axial load combinations and load factors for each foundation.

For axial compression loading, the drilled shafts at Pier 2 are designed to satisfy the LRFD criterion for Strength I, Extreme Event I, and Service I limit states. Axial load demands for each of these limit states are given in TABLE A-1.

TABLE A-2 SUMMARY OF GEOMATERIAL LAYER THICKNESSES AND GEOMATERIAL TYPES

Geomaterial Layer No.	Elevation, Top of Layer (ft)	Elevation, Bottom of Layer (ft)	Thickness (ft)	Depth Interval (ft), from Mudline	Subsurface Stratum	Geomaterial Type	
						Cohesionless Soil	Cohesive Soil
1	-29	-54	25	0-25	C	X	-
2	-54	-94	40	25-65	C	X	-
3	-94	-114	20	65 to 85	D	X	-
4	-114	-129	15	85 to 100	D	-	X
5	-129	-139	10	100 to 110	D	X	-
6	-139	-219.5	80.5	110 to 190.5	D and E	-	X
7	-219.5	-241.5	22	190.5 to 212.5	F	X	-
8	-241.5	-249	7.5	212.5 to 220	F	-	X

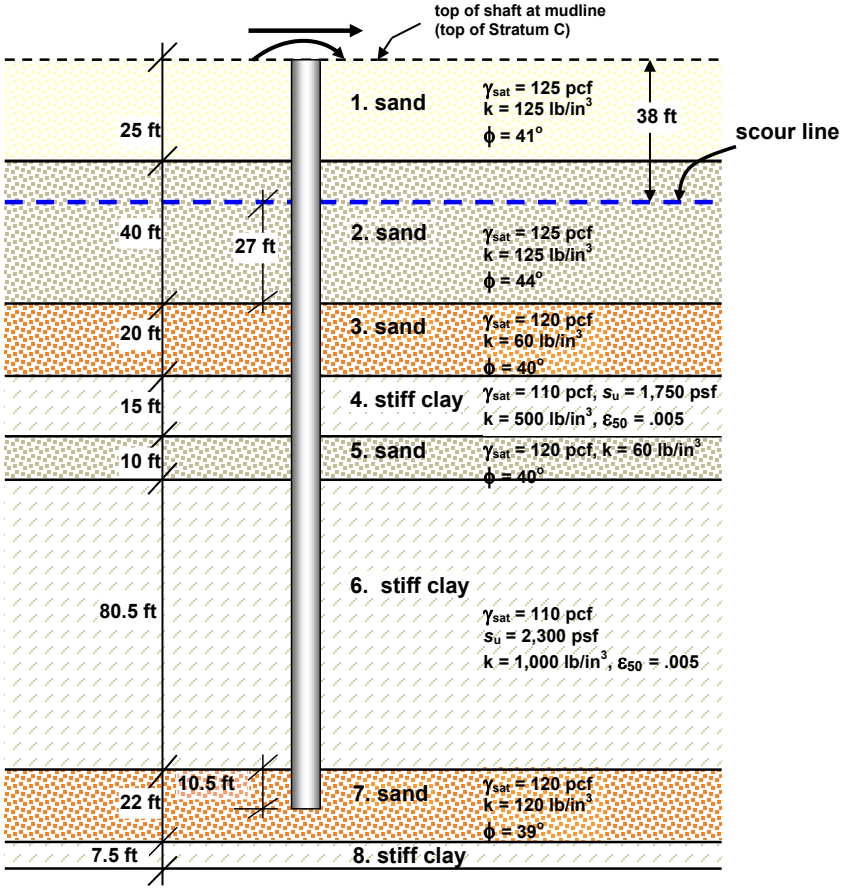


Figure A-4 Idealized Geomaterial Layer Profile for Drilled Shaft Design at Pier 2

Step 11.3 For each limit state, establish the loading mode(s) to be analyzed for overall foundation response (drained loading, undrained loading, or both) and assign corresponding geomaterial properties needed for evaluation of axial resistances for each geomaterial layer.

Cohesionless soil layers are analyzed for drained loading response, while cohesive soil layers are analyzed for undrained loading response. The corresponding strength properties required for calculating resistances are: effective stress friction angle (ϕ) for cohesionless soils and undrained shear strength (s_u) for cohesive soils. Table A-3 shows the calculation results for determination of soil friction angle for each of the cohesionless soil layers (Layers 1, 2, 3, 5 and 7). For each layer, the field N-values (N_{60}), depth, and vertical effective stress corresponding to each measurement are tabulated. Each field N-value is corrected for effective overburden stress to yield $(N_1)_{60}$ according to the following equation presented in Chapter 3:

$$(N_1)_{60} = N_{60}\left(\frac{p_a}{\sigma'_{vo}}\right)^n \qquad \text{A-1}$$

where p_a = atmospheric stress (2,116 psf), σ'_{vo} = vertical effective stress at the depth of the SPT N-value measurement, and n = 0.5 for sandy soils. The mean value of $(N_1)_{60}$ and corresponding coefficient of variation are then calculated for each layer. If the coefficient of variation is within the specified limit ($\leq$ 45%), the mean value of $(N_1)_{60}$ is then correlated to effective stress friction angle by the following equation, also presented in Chapter 3:

$$\phi' = 27.5 + 9.2\log\left[(N_1)_{60}\right] \qquad \text{A-2}$$

Step 11.4 For each drilled shaft, select trial lengths and diameters for initial analyses. As stated, the trial diameter is 8 ft and the trial shaft length is 201 ft from mudline to tip elevation (elevation -67 ft to elevation -268 ft).

Steps 11.5 and 11.6 Compute values of nominal side resistance for all geomaterial layers through which the trial shaft extends and the nominal base resistance at the trial tip elevation. Select appropriate resistance factors and calculate factored resistances.

Calculations of nominal side and base resistances are readily carried out using a spreadsheet or other computational desktop tool. Table A-4 shows the calculation results for Strength I limit state of this example. The table is divided into three areas: (A) side resistance in cohesionless soil layers; (B) side resistance in cohesive soil layers, and (C) base resistance. Example calculations are presented below for a single layer within each group.

A. Consider geomaterial Layer 3 for illustrating the beta method of computing side resistance of a cohesionless soil layer. The computations follow the equations presented in Chapter 10. The example is for Strength I limit state, therefore full scour is taken into account.

   Establish mean value of N_{60} and mean vertical effective stress σ'_v

   mean N_{60} = 30,mean σ'_v = 4,645 psf (no scour); mean σ'_v = 2,266 psf (full scour);

TABLE A-3 FRICTION ANGLE CORRELATIONS FOR COHESIONLESS SOIL LAYERS

Layer	Depth (ft)	σ'_v (psf)	Field N-value	Corrected N-value $(N_1)_{60}$	Mean $(N_1)_{60}$	Coefficient of variation	φ (degrees)
Layer 1	5	313.0	22	44	28	33%	41
	10	626.0	17	31			
	15	939.0	13	20			
	20	1252.0	22	29			
	25	1565.0	16	19			
Layer 2	30	1878.0	40	42	61	33%	44
	35	2191.0	77	76			
	40	2504.0	100	92			
	45	2817.0	91	79			
	50	3130.0	65	53			
	55	3443.0	74	58			
	60	3756.0	81	61			
	65	4069.0	35	25			
Layer 3	65	4069.0	35	25	21	23%	40
	70	4357.0	38	26			
	75	4645.0	21	14			
	80	4933.0	25	16			
	85	5221.0	32	20			
Layer 5	100	5,935	34	20	21	--	40
	105	6,223	38	22			
Layer 7	195	10,602	41	18	18	3%	39
	200	10,890	41	18			
	205	11,178	39	17			
	210	11,466	43	18			

Establish ϕ' by correlation to $(N_1)_{60}$

mean $(N_1)_{60} = 21$ (see Table A-3)

$$\phi' = 27.5 + 9.2 \log\left[\left(N_1\right)_{60}\right] = 27.5 + 9.2[\log(21)] = \underline{40 \text{ degrees}}$$

$\delta = \phi = 40^o$

Establish preconsolidations stress (σ'_p) by correlation to N_{60} and calculate overconsolidation ratio (OCR):

$\sigma'_p = p_a\,[0.47\,(N_{60})^m] = 2{,}116$ psf $[0.47\,(30)^{0.6}] = 7{,}654$ psf

$OCR = \sigma'_p / \sigma'_v = 7{,}654 / 4{,}645 = 1.65$

Calculate K_o using estimated values of OCR and ϕ'

$$K_o = (1 - \sin\phi')\,OCR^{\sin\phi'} = (1 - \sin 40^o)\,1.65^{\sin 40} = 0.49 \leq K_p$$

Calculate β and average nominal unit side resistance f_{SN}

$\beta = K \tan\delta = 0.49 \tan 40^o = 0.41$

$f_{SN} = \beta \times \sigma'_v = 0.41$ (2,266 psf) = 936 psf *note that σ'_v accounts for scour

Calculate Layer 3 nominal side resistance R_{SN}

$R_{SN} = \pi B\,\Delta z\,f_{SN} = \pi$ (8 ft) (20 ft) 932 lb/ft^2 = 470,684 lb

Select appropriate resistance factor and calculate factored resistance:

By Table A-5, the resistance factor for side resistance of a drilled shaft under axial compression by the β- method is 0.55. The factored side resistance of geomaterial Layer 3 therefore is:

$(\varphi_s\,R_{SN})_{i=3} = 0.55$ (470,684 lb) = 258,876 lb

B. Consider geomaterial Layer 4 for illustrating the computation of side resistance in a cohesive soil layer. In accordance with Chapter 10, the alpha method is used for undrained side resistance of shafts in cohesive soil.

Establish the mean value of soil undrained shear strength

Based on unconsolidated undrained (UU) triaxial tests, mean s_u = 1,750 psf

Transform the UU undrained shear strength to its equivalent CIUC value by Equation 10-17:

$$\frac{s_u(UC)}{s_u(CIUC)} = 0.893 + 0.513 \log\left(\frac{s_u(UC)}{\bar{\sigma}_{vo}}\right) = 0.893 + 0.513 \log\left(\frac{1{,}750\ psf}{2{,}114\ psf}\right) = 0.851$$

s_u(CIUC) = 1,750 psf / 0.851 = 2,057 psf

Calculate the value of the coefficient α:

$$\alpha = 0.30 + \frac{0.17}{\left(\frac{s_{u(CIUC)}}{p_a}\right)} = 0.30 + \frac{0.17}{\left(\frac{2,057}{2,114}\right)} = 0.47$$

Calculate layer nominal unit side resistance:

$f_{SN} = \alpha\ s_u = 0.47\ (2{,}057\ \text{psf}) = 976\ \text{psf}$

Calculate nominal layer side resistance:

$R_{SN} = \pi B\ \Delta z\ f_{SN} = \pi\ (8\ \text{ft})\ (15\ \text{ft})\ 967\ \text{lb/ft}^2 = 368{,}124\ \text{lb}$

Select applicable resistance factor and calculate Layer 4 factored side resistance:

Table 10-5 specifies a resistance factor of 0.45 for side resistance in compression by the α-method.

$(\varphi_s\ R_{SN})_{i=4} = 0.45\ (368{,}124\ \text{lb}) =$ **<u>165,656 lb</u>**

C. Base resistance is calculated for the trial design length of 201 ft, which places the tip in Layer 7, categorized as cohesionless soil. Applying the equations presented in Chapter 10, the nominal unit base resistance is given by:

$q_{BN} = 0.60\ N_{60} = 0.60\ (41) = 24.6\ \text{tsf} = 49.2\ \text{k/ft}^2$

in which N_{60} is the average field N-value (uncorrected) over a depth of two shaft diameters beneath the tip. Nominal base resistance is the product of unit base resistance and the cross-sectional area of the shaft:

$R_{BN} = \pi/4\ (8\ \text{ft})^2\ 49.2\ \text{k/ft}^2 = 2{,}473\ \text{kips}$

By Table 10-5, the resistance factor for base resistance in compression is 0.50. The factored base resistance is then given as:

$(\varphi_B\ R_{BN}) = 0.50\ (2{,}473\ \text{kips}) =$ **<u>1,236.5 kips</u>**

Table A-4 presents the calculated side resistances for each layer, and shows the summation of factored side resistance for the cohesionless layers, the cohesive layers, and factored base resistance. Note that the spreadsheet calculations may exhibit minor differences compared to those presented above as a result of rounding differences. The summation of factored resistances is given by:

$$\sum \varphi_i R_i = 1{,}087{,}053 + 1{,}181{,}955 + 1{,}236{,}531 = 3{,}505{,}539\ lb =$$ **<u>3,506 kips</u>**

Referring to Table A-1, the factored maximum axial compression force for Strength I limit state is 3,389 kips. The limit state check is given by:

3,389 kips < 3,506 kips

and the trial design therefore satisfies the Strength I limit state for geotechnical axial compression.

TABLE A-4 SPREADSHEET CALCULATIONS FOR AXIAL COMPRESSIVE RESISTANCE, STRENGTH I LIMIT STATE

Spreadsheet A: Side Resistance, Cohesionless Soil Layers

Geomaterial Layer No.	1	2	3	5	7	Σ
Thickness, ft (after scour)	0	27	20	10	10.5	--
Average φ	41	44	40	40	39	--
Existing Ground: Mean Vertical Effective Stress, σ'_v (psf)	783	2,817	4,645	6,223	10,645	--
After Scour: Mean Vertical Effective Stress, σ'_v (psf)	0	845	2,266	3,844	8,266	--
Mean N-value (N_{60})	17	75	30	36	41	--
σ_P (psf)=0.47$(N_{60})^m$	5,444	13,263	7,654	8,539	9,232	--
OCR	7.0	4.7	1.6	1.4	1.0	--
$K=K_O$	1.23	0.90	0.49	0.44	0.37	--
B=K tanφ	1.07	0.87	0.41	0.37	0.30	--
Nominal Unit Side Resistance, $f_{SN}=\beta\ \sigma'_v$	0	731	936	1,412	2,481	--
Layer Nominal Side Resistance, R_{SN} (lb)	0	496,072	470,684	354,897	654,807	--
Side Resistance Factor, φ_S	NA	0.55	0.55	0.55	0.55	--
Layer Factored Side Resistance, $\varphi_S R_{SN}$ (lb)	0	272,840	258,876	195,193	360,144	1,087,053

Spreadsheet B: Side Resistance, Cohesive Soil Layers

Geomaterial Layer No.	4	6	Σ
Thickness, ft	15	80.5	--
Average CIUC Undrained Shear Strength (ps)	2,057	2,523	--
Coefficient α	0.47	0.44	--
Nominal Unit Side Resistance, fSN= α c_u	976	1,116	--
Layer Nominal Side Resistance, R_{SN} (lb)	368,124	2,258,442	--
Side Resistance Factor, φ_S	0.45	0.45	--
Layer Factored Side Resistance, $\varphi_S R_{SN}$ (lb)	165,656	1,016,299	1,181,955

Spreadsheet C: Base Resistance, Layer 7, Cohesionless Soil

Geomaterial Layer No.	7
Mean N-value for 2-Diameters Below Tip	41
Nominal Unit Side Resistance = 0.6 N_{60} (tsf)	24.6
Nominal Base Resistance, R_{BN} (lb)	2,473,062
Base Resistance Factor, φ_B	0.50
Factored Base Resistance, $\varphi_B R_{BN}$ (lb)	1,236,531

Summation of Factored Resistances = 3,506 kips

A similar analysis is conducted for a limit state check of Extreme Event I (earthquake). In this case, zero scour is assumed, and the full soil profile shown in Figure A-4 is taken into account. The spreadsheet calculations, presented in Table A-5, are essentially the same as those made for the Strength I limit state, except that all resistance factors are equal to 1.00 and the absence of scour results in higher values of vertical effective stress, resulting in higher side resistances in the cohesionless soil layers. As shown at the bottom of Table A-5, the summation of factored resistances equals 9,932 kips. From Table A-1, the maximum factored axial force effect for Extreme Event I is 2,823 kips. The limit state check is given by:

2,823 kips < 9,932 kips

The factored resistance is substantially greater than the factored axial demand and the trial design therefore easily satisfies the Extreme Event I limit state for axial compression.

TABLE A-5 SPREADSHEET CALCULATIONS OF AXIAL COMPRESSIVE RESISTANCE FOR EXTREME EVENT I LIMIT STATE

Spreadsheet A: Side Resistance, Cohesionless Soil Layers

Geomaterial Layer No.	1	2	3	5	7	Σ
Thickness, ft (after scour)	25	40	20	10	10.5	--
Average φ	41	44	40	40	39	--
Existing Ground: Mean Vertical Effective Stress, σ'_v (psf)	783	2,817	4,645	6,223	10,645	--
After Scour: Mean Vertical Effective Stress, σ'_v (psf)	783	2,817	4,645	6,223	10,645	--
Mean N-value (N_{60})	17	75	30	36	41	--
σ_P (psf)=0.47$(N_{60})^m$	5,444	13,263	7,654	8,539	9,232	--
OCR	7.0	4.7	1.6	1.4	1.0	--
$K=K_O$	1.23	0.90	0.49	0.44	0.37	--
B=K tanφ	1.07	0.87	0.41	0.37	0.30	--
Nominal Unit Side Resistance, $f_{SN}=\beta\ \sigma'_v$	835	2,437	1,919	2,286	3,195	--
Layer Nominal Side Resistance, R_{SN} (lb)	524,776	2,449,741	964,754	574,507	843,239	--
Side Resistance Factor, φ_S	1.00	1.00	1.00	1.00	1.00	--
Layer Factored Side Resistance, $\varphi_S R_{SN}$ (lb)	524,776	2,449,741	964,754	574,507	843,239	4,832,242

Spreadsheet B: Side Resistance, Cohesive Soil Layers

Geomaterial Layer No.	4	6	Σ
Thickness, ft	15	80.5	--
Average CIUC Undrained Shear Strength (ps)	2,057	2,523	--
Coefficient α	0.47	0.44	--
Nominal Unit Side Resistance, fSN= α c_u	976	1,116	--
Layer Nominal Side Resistance, R_{SN} (lb)	368,124	2,258,442	--
Side Resistance Factor, φ_S	1.00	1.00	--
Layer Factored Side Resistance, $\varphi_S R_{SN}$ (lb)	368,124	2,258,442	2,626,566

Spreadsheet C: Base Resistance, Layer 7, Cohesionless Soil

Geomaterial Layer No.	7
Mean N-value for 2-Diameters Below Tip	41
Nominal Unit Side Resistance = 0.6 N_{60} (tsf)	24.6
Nominal Base Resistance, R_{BN} (lb)	2,473,062
Base Resistance Factor, φ_B	1.00
Factored Base Resistance, $\varphi_B R_{BN}$ (lb)	2,473,062

Summation of Factored Resistances = 9,932 kips

The final geotechnical limit state to be evaluated for axial loading is Service I. The bridge structural designer has established a tolerable settlement of 2.0 inches for single shafts at Pier 2. Applying the first-order approximation involving the use of normalized displacement curves for single drilled shafts in soil (as shown in Figure 10-40) the resistance corresponding to a downward displacement of 2 inches can be estimated as follows:

"Failure Threshold" $L_2 = R_{SN} + R_{BN} = (1{,}976.5 + 2{,}626.6) + 2{,}473.1 = 7{,}076.2$ kips

Normalized displacement = $\delta_{tolerable}$ / B = 2 in / 96 in (100%) = 2.1%

Entering Figure 10-40, reproduced below as Figure A-5, with a normalized displacement of 2.1% yields a normalized test load of 78%.

"Test Load" = $R_{service}$ + W = 0.78 (7,076.2) = 5,519 kips

$R_{service}$ = 5,519 – W = 5,519 – 885.0 = 4,634 kips

The limit state check is then written as:

2,835.4 kips < 4,634 kips

or, the factored Service I axial demand is less than the axial resistance developed at the tolerable displacement, and therefore the trial design satisfies Service I limit state

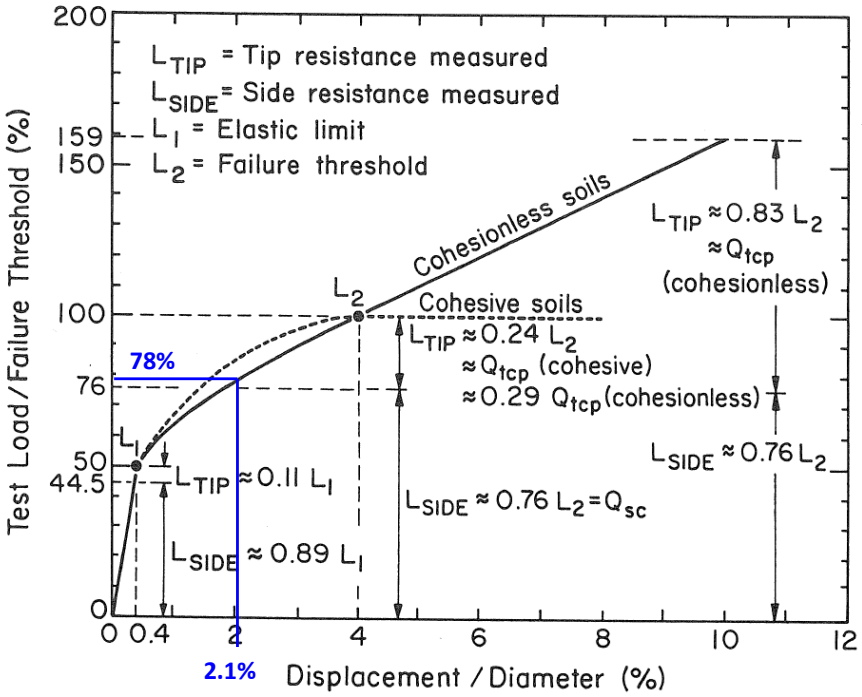


Figure A-5 Normalized Load-Displacement Curve for Analysis of Service Limit State

Step 10: Establish Minimum Diameter and Depth for Lateral Loads

As described in Chapter 9, drilled shafts are designed to withstand lateral loading by selecting the dimensions (diameter and depth) and structural properties to satisfy LRFD limit state checks on the geotechnical strength and structural strength for all applicable strength and extreme event limit states. For service limit states, the shaft must develop sufficient resistance at tolerable lateral displacements specified by the structural designer. Figure 9-12 outlines a design process for lateral loading, in which each step is identified as a substep of Block 10 of the overall process.

Step 10.1 Define the detailed subsurface profiles for each lateral load case, including scour. The profile shown in Figure A-4, developed for axial loading, is also applied to the lateral load cases considered in this example. Soil properties used for lateral load analysis by the *p-y* method are also shown in the profile (Figure A-4). For Strength I and Service I limit states, full scour of 38 ft is assumed, while for Extreme Event I zero scour is assumed.

Step 10.2 Select Trial Length and Diameter. The trial dimensions established in the axial load analysis, 8-ft diameter, 201-ft deep shafts, are checked for each limit state under lateral loading. In addition, minimum depths needed to satisfy lateral loading requirements are evaluated.

Step 10.3 Analyze the geotechnical strength limit state using factored loads.

As described in Section 12.3.3.1, the geotechnical strength under lateral loading is evaluated by conducting a "pushover analysis" in which force effects are applied to the head of the shaft in various multiples up to and exceeding the factored force effects. For each load multiple, lateral head deflection is calculated and the load-deflection curve must exhibit stable behavior up to the maximum load multiples. In addition, lateral deflection at the maximum load must be less than 10% of the shaft diameter. The concrete shaft is modeled in this analysis as a linear elastic beam of modulus equal to that of the concrete and moment of inertia taken as that of the uncracked section.

The *p-y* method of analysis is used to illustrate the pushover analysis for Strength I and Extreme Event I limit states. For the Strength I limit state, the factored lateral demands presented in Table A-1 are considered. Lateral and moment demands are given for both longitudinal and transverse directions. For the *p-y* analysis, the resultant lateral and moment force effects are resolved as follows:

$$V = \sqrt{\left(V_{longitudinal}\right)^2 + \left(V_{transverse}\right)^2} = \sqrt{(36.98)^2 + (9.63)^2} = 38.21\ kips$$

$$M = \sqrt{\left(M_{longitudinal}\right)^2 + \left(M_{transverse}\right)^2} = \sqrt{(1{,}686.87)^2 + (221.68)^2} = 1{,}701.37\ kip - ft$$

The minimum axial compressive force given in Table A-1 for Strength I (2,673.42 kips) is also applied, in combination with the factored shear and moment. Drilled shaft properties used in the LPILE analysis are summarized in Table A-6.

TABLE A-6 DRILLED SHAFT PROPERTIES FOR PUSHOVER ANALYSIS

Drilled Shaft Property	Value
Diameter	96 inches
Length	2,412 inches
Distance from pile top to ground surface	456 inches
Moment of inertia	4,169,220 in^4
Area	7,238 in^2
Modulus of elasticity	4,000,000 lb/in^2

The soil layers and properties are as shown in Figure A-4, and the Strength I analysis incorporates full scour to a depth of 38 ft below the top of the shaft (hence a distance of 456 inches from top of pile to ground surface). The pushover analysis is conducted by applying shear and moment in increments up to maximum values of 1/φ times the factored demands, where φ = resistance factor. For Strength I, φ = 0.67, which is equivalent to multiplying the factored demands by 1.5. For this analysis, the lateral load and moment were applied in multiples of 0.25 up to 1.5 times the factored values. The actual load combinations and resulting lateral head deflection values from the LPILE output are summarized in Table A-7.

TABLE A-7 LOADING AND COMPUTED HEAD DEFLECTIONS, PUSHOVER ANALYSIS OF TRIAL SHAFT, STRENGTH I

Load Increment	Multiple of Factored Demands	Shear Force, V (lbs)	Moment, M (in-lbs)	Axial Force, Q (lbs)	Computed Lateral Head Deflection, y (inches)
1	0.25	9,553	5,104,121	1,836,420	0.175
2	0.50	19,107	10,208,242	1,836,420	0.350
3	0.75	28,660	15,312,363	1,836,420	0.526
4	1.00	38,213	20,416,484	1,836,420	0.702
5	1.25	47,767	25,520,605	1,836,420	0.879
6	1.50	57,320	30,624,726	1,836,420	1.056

Figure A-6 shows the computed lateral deflection versus depth curves for the six load combinations. Lateral deflection reduces to zero at a depth of approximately 60 ft, which is 26 ft below the ground surface (below scour line). This observation suggests that the shaft trial length of 201 ft is greater than necessary for geotechnical strength under lateral loading; however this length is required for axial design. The shaft lengths are therefore controlled by axial loading, not lateral loading.

In Figure A-7, the lateral head deflection is plotted against the load multiplier for each of the six cases. The curve for the trial shaft length of 201 ft exhibits stable behavior up through 1.5 times the factored force effects, and maximum deflection of 1.06 inches is well within 10 percent of the shaft diameter. Based on the pushover analysis, the trial design therefore satisfies the geotechnical Strength I limit state criterion.

To illustrate the minimum shaft depth required to provide adequate pushover stability, additional *p-y* analyses were conducted with all parameters held constant except shaft length. For a shaft length of 68 ft (corresponding to 30 ft of embedment below the scour line), the load-deflection behavior is still approximately linear and stable up to 1.5 times the factored force effects. However, for a shaft length of 62 ft (24 ft of embedment), the load deflection curve shows a nonlinear trend at higher loads and a deflection of 4 inches at 1.5 times the factored force effects, suggesting the onset of instability against overturning. For a shaft length shorter than 62 ft, the program will not converge to a solution, indicating this is the minimum depth for stability against pushover.

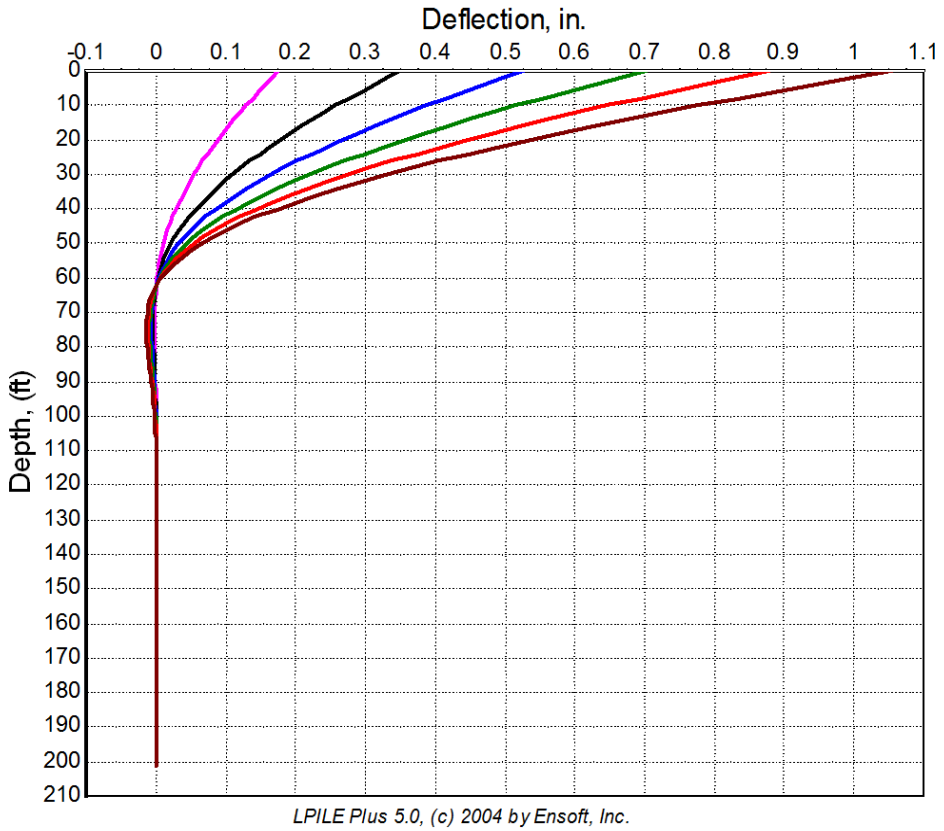


Figure A-6 Curves of Lateral Deflection versus Depth from Pushover Analysis of Trial Shaft

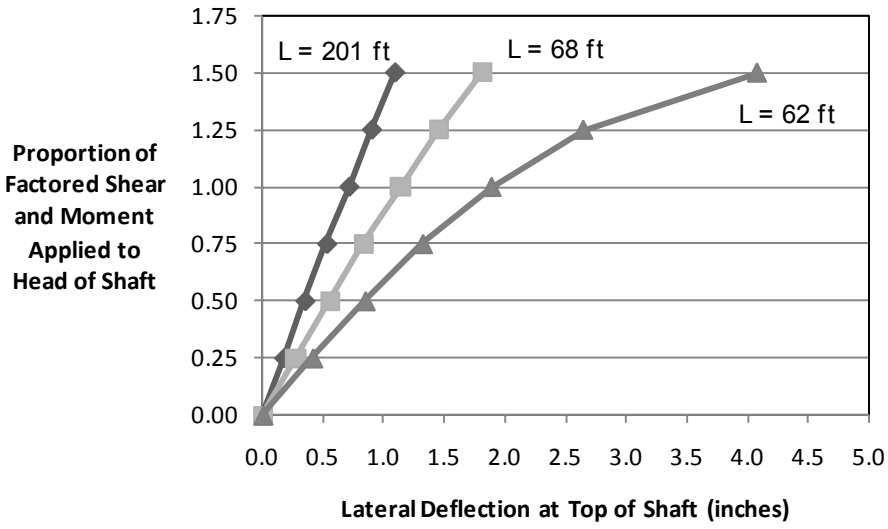


Figure A-7 Results of Pushover Analyses for Strength I Limit State

A similar approach is applied to evaluation of the Extreme Event I limit state. The soil profile shown in Figure A-4, but with no scour, is used to define the soil layers. In accordance with AASHTO design code (2017) two load cases are analyzed. The factored demands in the longitudinal direction are combined with 30 percent of the factored demands in the transverse direction to form the first load case, while the

second case consists of the demands in the transverse direction combined with 30 percent of the factored demands in the longitudinal direction. The first case is critical for pushover and is presented herein. Referring to the factored demands in Table A-1 for Extreme Event I, the factored shear and moment used in the analyses are given by:

$$V = \sqrt{\left(V_{longitudinal}\right)^2 + \left(\frac{1}{3}\, V_{transverse}\right)^2} = \sqrt{(144.52)^2 + \left(\frac{290.48}{3}\right)^2} = 173.96\ kips$$

$$M = \sqrt{\left(M_{longitudinal}\right)^2 + \left(\frac{1}{3} M_{transverse}\right)^2} = \sqrt{(7{,}776.42)^2 + \left(\frac{3{,}212.32}{3}\right)^2} = 7{,}849.79\ kip - ft$$

The minimum factored axial compressive force of 2,810.26 kips is also applied. The drilled shaft is treated as a linearly elastic beam-column with properties as given in Table A-6, except that the distance from the top of the shaft to the ground surface in this case is zero, corresponding to no scour.

The pushover analysis is performed by applying the shear (V) and moment (M) in increments up to maximum values of 1/φ times the factored demands, where φ = resistance factor. For Extreme Event I, φ = 0.80, which is equivalent to multiplying the factored force effects by 1.25. The shear and moment were applied in multiples of 0.25 up to 1.25 times the factored values. The load combinations and resulting lateral head deflection values from the LPILE output are summarized in Table A-8.

Figure A-8 shows the lateral head deflection versus the load multiplier for each of the five load increments, for the trial shaft length of 201 ft and also for shorter lengths to establish the minimum required depth for pushover stability. The trial design length (201 ft) exhibits stable, approximately linear behavior up through 1.25 times the factored demands, and maximum deflection of 0.51 inches is well within an acceptable range. The trial design therefore provides adequate lateral geotechnical strength for the Extreme Event I limit state.

TABLE A-8 Loading and Computed Head Deflections,
Pushover Analysis of Trial Shaft, Extreme Event I

Load Increment	Multiple of Factored Force Effects	Shear Force, V (lbs)	Moment, M (in-lbs)	Axial Force, Q (lbs)	Computed Lateral Head Deflection, y (inches)
1	0.25	43,490	23,549,382	2,810,260	0.095
2	0.50	86,980	47,098,763	2,810,260	0.191
3	0.75	130,470	70,648,145	2,810,260	0.289
4	1.00	173,960	94,197,526	2,810,260	0.396
5	1.25	217,450	117,746,908	2,810,260	0.512

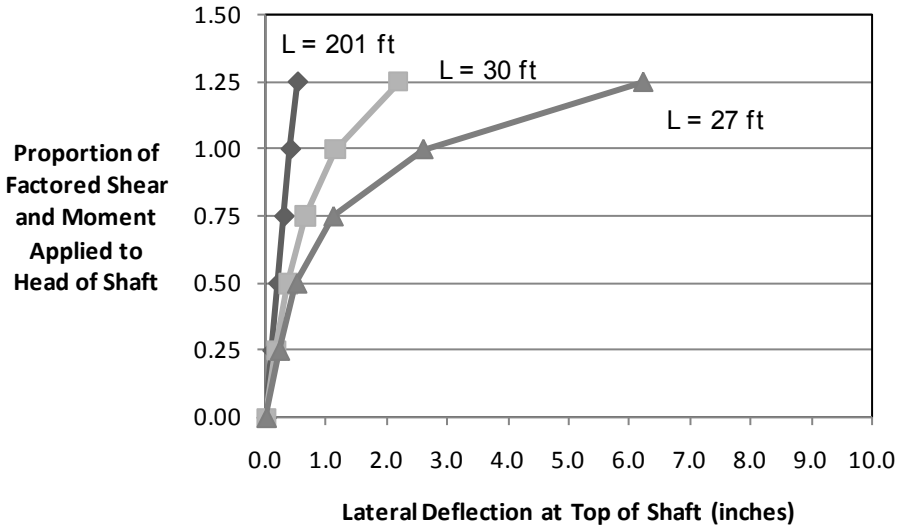


Figure A-8 Results of Pushover Analysis for Extreme Event I

For a shaft length of 30 ft (fully embedded), the load-deflection behavior becomes nonlinear, suggestion that an unstable length is being approached, and a shaft of 27 ft in length shows deflection at the overstress load level (multiplier = 1.25) exceeding 6 inches. If the shaft length is decreased to 26 ft, the program will not converge. This exercise indicates that 30 ft is the approximate minimum shaft length required to satisfy lateral geotechnical strength for Extreme Event I.

Step 10.4 Analyze preliminary structural strength limit state for flexure using factored loads. This step provides a preliminary check of moment resistance of the trial design, which is then compared to maximum bending moments obtained from the pushover analyses, in which the shaft is treated as a simple linearly elastic beam. Nominal moment resistance (M_N) is approximated by (see Chapter 9, Section 9.3.2.2):

for $\rho = 1\%$ M_N (ft-k) = 27 D^3= 27 $(8ft)^3$ = 13,824 ft-k= 165,888 in-k

for $\rho = 1.5\%$ M_N (ft-k) = 40 D^3= 40 $(8ft)^3$ = 20,480 ft-k= 245,760 in-k

From the Strength I limit state pushover analysis, M_{max} = 40,981 in-k, and for the Extreme Event I pushover analysis, M_{max} = 107,810 in-k. Considering that resistance factors for flexure range from 0.75 to 0.90, the 8-ft diameter shafts with ρ in the range considered above (1.0% – 1.5%) will provide sufficient flexural resistance. There is no need at this point to consider larger diameters. Complete design for structural strength incorporating the nonlinear bending stiffness of the shaft, is considered in Block 12 of the overall design process.

Step 10.5 Analyze service limit state (deformations) using unfactored loads.

In consultation with the bridge structural engineer, a maximum tolerable lateral deformation of ½ inch at the scour line (38 ft below the top of the shaft) is established for serviceability. The trial design is analyzed by the *p-y* method of analysis, subject to the Service I factored demands. Since service load factors specified by AASHTO currently are 1.00, these are essentially unfactored load demands.

Referring to Table A-1, the factored shear and moment demands used in the Service I analyses are given by:

$$V = \sqrt{\left(V_{longitudinal}\right)^2 + \left(V_{transverse}\right)^2} = \sqrt{(54.65)^2 + (34.59)^2} = 64.68\ kips$$

$$M = \sqrt{\left(M_{longitudinal}\right)^2 + \left(M_{transverse}\right)^2} = \sqrt{(2{,}647.17)^2 + (317.24)^2} = 2{,}666.11\ kip - ft$$

The minimum factored axial compressive force of 1,836.42 kips is also applied.

The program input used for the pushover analysis of the 201-ft trial shaft length is adjusted to the service limit state loads and additional input is required to characterize the nonlinear structural properties of the reinforced concrete shaft. The shaft is specified to be a circular, nonlinear reinforced concrete member, 96 inches diameter, reinforced with 40 #14 bars, and with 6-inches of concrete cover. This provides a steel area ratio (ρ) of approximately 1.25 percent of the concrete gross cross-sectional area. Concrete compressive strength q_c = 4,000 psi and yield strength of the steel reinforcement bars f_y = 60,000 psi. Figure A-9 shows the deflection versus depth curve from *the p-y* analysis. At a depth of 38-ft (scour line), the lateral deformation is 0.49 inches, just within the specified tolerance of ½ inch. The trial design therefore satisfies the Service I limit state for lateral deformation.

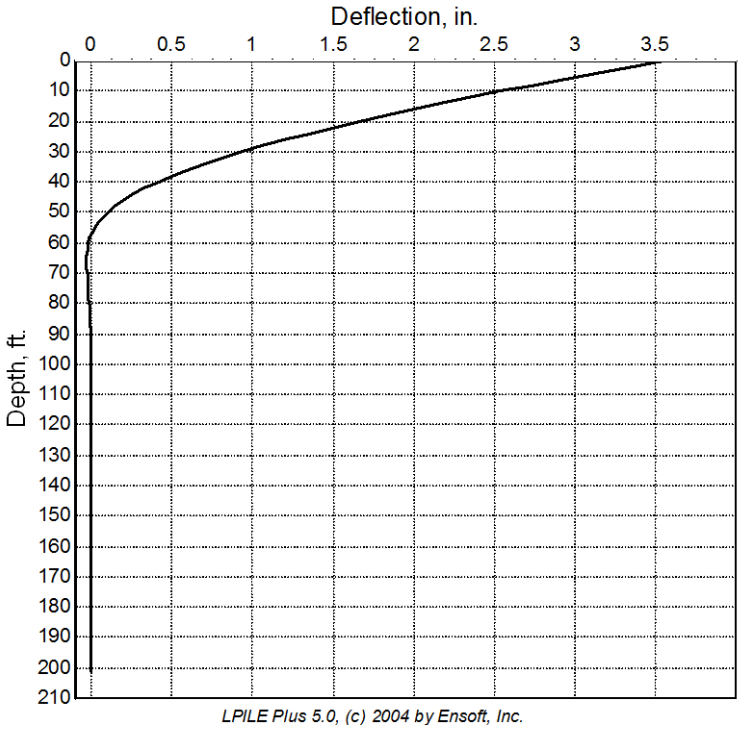


Figure A-9 Deflection versus Depth for Drilled Shaft Under Service I Force Effects

Substep 10.6 Define minimum length and diameter based on analyses

If lateral loading only is considered, the minimum length is approximately 65 ft, for 8-ft diameter shafts, based on the Strength I limit state, or pushover analysis. Based on axial load considerations, the minimum depth is 201 ft.

Step 12: Finalize Structural Design of the Shafts and Connection to Structure

In Step 10.4, above, it was shown that 8-ft diameter shafts with steel reinforcement ratios (ρ) in the range of 1 to 2 percent should be sufficient to satisfy structural strength requirements due to flexure. In Step 10.5, analysis shows that $\rho = 1.25$ percent is just sufficient to satisfy the service limit state requirement for ½ inch lateral displacement at the scour line. In this section, the trial design consisting of 8-ft diameter, 201-ft long shafts with $\rho = 1.25$ percent will be checked for structural resistance, for Strength I and Extreme Event I limit states. The procedure described herein follows the steps for structural design presented in Chapter 12 of this manual.

Step 12.1 Determine the factored force demands at the top of the drilled shaft.

Factored top-of-shaft demands at Pier 2 are given in Table A-1 for the Strength I and Extreme Event I limit states, based on structural modeling of the bridge and foundations as described previously

Step 12.2 Check trial design for axial resistance, *i.e.*, check whether the factored axial demand is well within the factored nominal axial resistance $\varphi(P_n)$ of the shaft using Equation (12-6).

Referring to Chapter 12, the factored axial resistance, given the trial shaft properties, is given by:

$$P_r = \varphi P_n = \varphi\beta\,[0.85 f_c'\,(A_g - A_s) + A_s f_y] = 0.75(0.85)\,[0.85 \times 4(7{,}238 - 90) + 90(60)]$$

$$P_r = 18{,}936.8 \text{ kips} >> 3{,}389.5 \text{ kips}$$

The factored axial demand (3,389.5) is well within the factored axial compressive resistance (18,936.8 kips) and the trial design therefore satisfies the structural Strength I limit state for compression.

Step 12.3 Analyze the shaft under lateral/moment loading to establish critical sections for moment and shear.

In this step, *p-y* analysis is carried out using top of shaft demands in combination with the nonlinear model of the reinforced concrete shaft. Graphs of moment versus depth and shear versus depth are generated and the program output file is inspected to determine maximum values of moment and shear demand and the corresponding depths at which they occur. Figure A-10 shows the resulting moment and shear diagrams for Strength I, and Table A-9 summarizes the critical values of moment, shear, and critical depths for Strength I and Extreme Event I force effects. The two cases analyzed for Strength I correspond to the minimum and maximum values of factored compression.

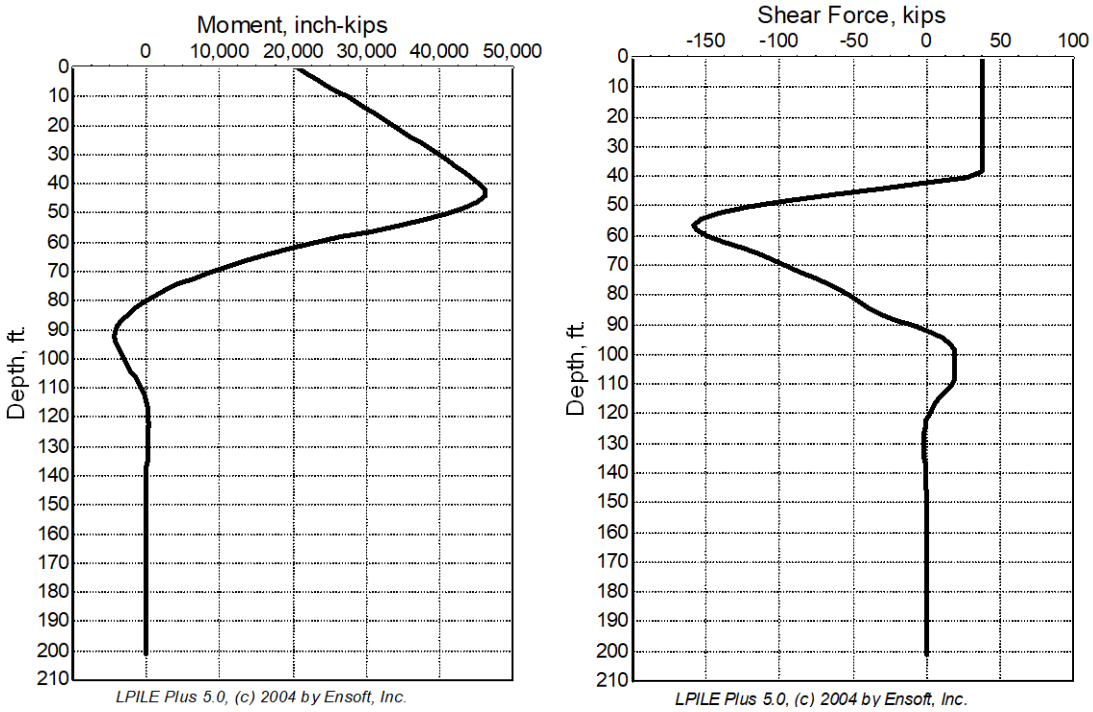


Figure A-10 Moment and Shear versus Depth, Drilled Shaft Under Strength I Force Effects

TABLE A-9 SUMMARY OF CRITICAL MOMENTS AND SHEAR FOR STRUCTURAL DESIGN

Limit State	M_{max} in-lb	M_{max} ft-kips	Depth of M_{max} inches	Depth of M_{max} ft	V_{max} (kips) lbs	V_{max} (kips) kips	Depth of V_{max} inches	Depth of V_{max} ft	P (kips)
Strength I	46,344,639	3,862	506.52	42.2	-158,218	-158.2	675.5	56.3	3,389.5
	44,647,327	3,721	506.52	42.2	-150,687	-150.7	675.4	56.3	2,673.4
Extreme Event I	1.08E+08	8,966	96.48	8.0	-425.6	-425.6	265.3	22.1	2,823.0

Step 12.4 Treating the drilled shaft as a beam-column, determine the required amount and distribution of longitudinal steel required for the section to resist the combined axial load and moment demands.

This step requires the designer to develop an interaction diagram, a curve defining the combination of axial compression (P) and moment (M) that defines the strength of a reinforced concrete beam column, as described in Chapter 12. For this example, *p-y* analyses were performed with the trial shaft dimensions and nonlinear bending properties to determine the moment required to achieve the strength limit state for a range of axial compression values from zero to the nominal crushing strength of 29,703 kips. The trial design consists of 4,000 psi concrete, 40 # 14 bars with f_y = 60 ksi, giving a steel ratio of 1.25%. The resulting P-M interaction diagram is shown in Figure A-11 as the nominal (or unfactored) interaction curve. These values are then factored according to AASHTO specifications as described in Section 12.7 of this manual. For axial strain values up to 0.002, compression controls and the resistance factor is taken as 0.75. For strain in the extreme tension fiber equal to or exceeding 0.005, the resistance factor is taken as 0.90. Between these limits of strain, the resistance factor is interpolated linearly between 0.75 and 0.90. Factored resistances are computed by multiplying both P_n and M_n by the appropriate single value of φ. The resulting factored interaction curve determined in this way is also shown in Figure A-11.

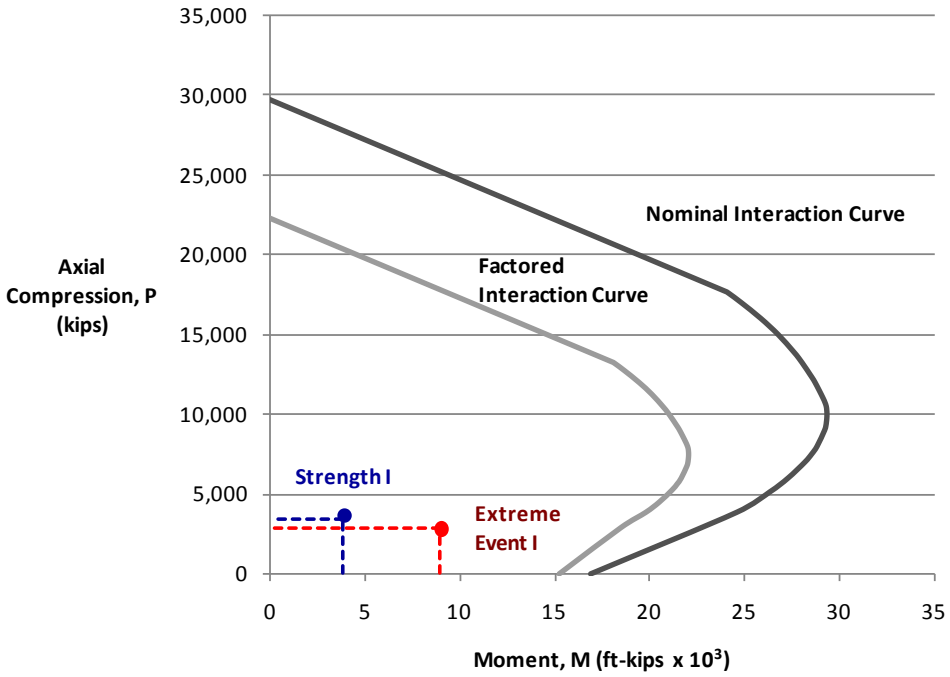


Figure A-11 P-M Interaction Diagram for Trial Shaft Structural Analysis

Limit state checks for flexure are then carried out by plotting the points representing the combination of factored axial and moment demands for each applicable limit state on the P-M interaction diagram. These points are shown on Figure A-11 for the Strength I (blue) and Extreme Event I (red) limit states. The values of factored demands (P and M) are those given in Table A-9. The LRFD requirement is that the P-M points lie within the factored interaction curves. For the Strength I limit state, the blue point must lie within the curve labeled "Factored Interaction Curve". For Extreme Event I, the resistance factors are taken equal to 1.00, which means that the factored resistances are equivalent to the nominal resistances. The point labeled Extreme Event I must therefore lie within the curve labeled "Nominal Interaction Curve". In this case, both points are well within the respective interaction curves, and therefore the trial structural design is satisfactory for flexural resistance for both Strength I and Extreme Event I limit states.

Step 12.5 Design for shear: check whether the concrete section has adequate shear resistance with the code-specified minimum transverse reinforcement; select appropriate ties or spirals according to AASHTO and ensure the spacing between reinforcement satisfies requirements for constructability.

First, determine whether the factored shear demand (425.6 kips from Table A-9 for Extreme Event I) exceeds one-half of the factored shear resistance provided by the concrete portion of the shaft cross section. By Eq. 12-15:

$$V_c = 0.0632\lambda\sqrt{f_c'}\, b_v d_v = 0.0632\sqrt{4\ ksi}(96\ in)(66.98\ in) = 812.8\ kips$$

$0.5\varphi V_c = 0.50\ (0.90)\ 812.8\ \text{kips} = 365.7\ \text{kips}$

Since the factored shear demand of 425.6 kips exceeds one-half of the factored shear resistance of the concrete, the code-specified minimum must be met. By Eq. 12-5 the required minimum is:

$$A_v(in^2) \geq 0.0316\,\lambda\sqrt{f'_c}\,\frac{b_v s}{f_y} = 0.0316\,(1.0)\sqrt{4\ ksi}\,\frac{96\ in(s)}{60\ ksi} = 0.101\ x\ s$$

where s = vertical spacing of the transverse reinforcement. Initially assume a spacing of 6 inches to meet the minimum clear space of 5 inches. In that case, A_v = 0.606 in^2. The minimum bar size meeting the required cross-sectional area is No. 8 with A_v = 0.785 in^2. Proceed with a trial design consisting of No. 8 hoops on a 6-inch spacing.

The next step is to evaluate the factored shearing resistance of the section, assuming #8 hoops on a 6-inch spacing. The nominal shearing resistance (V_n) is given by the smaller of the following two values:

$V_n = V_c + V_s$ or $V_n = 0.25\,f_c'\,b_v d_v$

in which V_c = nominal shear resistance provided by the concrete and V_s = nominal shearing resistance provided by the transverse reinforcement steel. In the second expression, the dimensions b_v and d_v refer to the width and effective depth of a rectangular cross-section, and the product b_v x d_v represents the cross-sectional area that is effective in resisting shear. For a circular cross-section, these parameters can be approximated by:

$$b_v = drilled\ shaft\ diameter, D = 96\ in$$

$$d_v = 0.9\left(\frac{D}{2} + \frac{D_r}{\pi}\right) = 0.9\left(\frac{96}{2} + \frac{83}{\pi}\right) = 66.98\ in$$

For a 6-in pitch the angle of the spiral to the vertical is given by:

$\alpha = \cos^{-1}$ [6 in / (π x 84 in)] = 88.7 degrees

By Eqs. 12-15 and 12-17:

$$V_c = 0.0632\sqrt{f_c'}\,b_v d_v = 0.0632\sqrt{4\ ksi}\,(66.98\ in)(96\ in) = 812.8\ kips$$

$$V_s = \frac{A_v f_y d_v}{s} = \frac{(0.785\ in^2)(60\ ksi)(66.98\ in)}{6\ in} = 525.8\ kips$$

$V_n = V_c + V_s$ = 812.8 + 525.8 = 1,338.6 kips

As a check: $0.25\,f_c'\,b_v d_v$ = 0.25 (4 ksi) 96 in (66.98 in) = 6,430 kips > 1,339 kips

The factored shear resistance of the trial design is therefore:

$V_r = \varphi V_n$ = 0.90 (1,338.6 kips) = 1,204.7 kips

Limit state checks are then carried out by comparing the factored shear demand (Table A-9) to the factored shear resistance:

For the Strength I limit state: 158.2 kips < 1,204.7 kips

For the Extreme Event I limit state:425.6 kips < 1,204.7 kips

The factored shear resistance easily exceeds factored shear demand for both limit states. The trial design consisting of No. 8, Grade 60 hoops on a 6-inch spacing therefore satisfies the required minimum transverse reinforcement and meets the shear demand for strength and extreme event limit states.

As stated in Chapter 12, AASHTO specifications require that the minimum amount of transverse steel extend from the top of the drilled shaft to a depth of at least three diameters below the calculated depth of moment fixity. Based on the *p-y* analyses of the trial shaft for Strength I and Extreme Event I, the maximum depth of moment fixity is 110 ft below the existing mudline elevation (see Figure A-10). Three shaft diameters (24 ft) establishes a depth of 134 ft below mudline elevation as the depth to which the minimum transverse reinforcement is required. Also considering that the drilled shafts extend from the existing mudline elevation (-29 ft) upward to the connection with the pier columns at elevation + 3.64 ft, the total shaft length over which the minimum traverse steel is required is approximately 167 ft. The spiral size and pitch required to meet the minimum transverse steel requirement has been calculated above, and consists of: No. 8, Grade 60 spiral at s = 6.0 inch. Below the depth corresponding to three diameters below moment fixity, the transverse reinforcement can be reduced to a size and spacing that will provide a stable reinforcing cage for handling and constructability, for example No. 5 hoops or spiral on 12-inch pitch.

The clear spacing between #8 hoops at 6-inch spacing is 5 inches. If the No. 14 longitudinal bars are bundled radially in groups of two, the circumferential clear space between bundles is approximately 11 inches. Both values of clear spacing meet the recommended minimum value of 5 inches (Chapter 6).

Below the depth at which the drilled shafts behave as columns, which can be taken as the depth below which bending moment is zero, or approximately 115 ft below existing mudline, the minimum recommended reinforcement ratio of 0.5 percent can be used. If one bar of each two-bar bundle of #14 bars is discontinued below this depth, the steel ratio is approximately 0.6 percent, a practical solution and one that provides continuity in the reinforcement cage.

Connection to Pier Columns. A construction joint is located at the connection between the drilled shafts and the pier columns at elevation +5.0 ft, which is approximately 2.4 ft above mean high water level (M.H.W.). Figure A-12 shows a partial elevation of the drilled shafts at Pier 2 and selected cross-sections of the drilled shafts for the cased (Section A-A) and uncased (Section B-B) lengths. The column and shaft reinforcement are the same for the drilled shafts and pier columns (40 #14 bars in bundles of two). Mechanical couplers are specified for splicing the longitudinal bars at the construction joint, with connections between individual bars within each 2-bar bundle to be staggered. The transverse steel for the drilled shafts, consisting of bundled #8 hoops at s = 6 inches, also extends into the pier columns and is continued 5 ft into the pier cap. The interior bar of each 2-bar bundle is to be straight and also embedded 5 ft into the pier cap. Following construction of the superstructure, the permanent casing is to be cut from the top, to 2 ft below the mean low water elevation (M.L.W.).

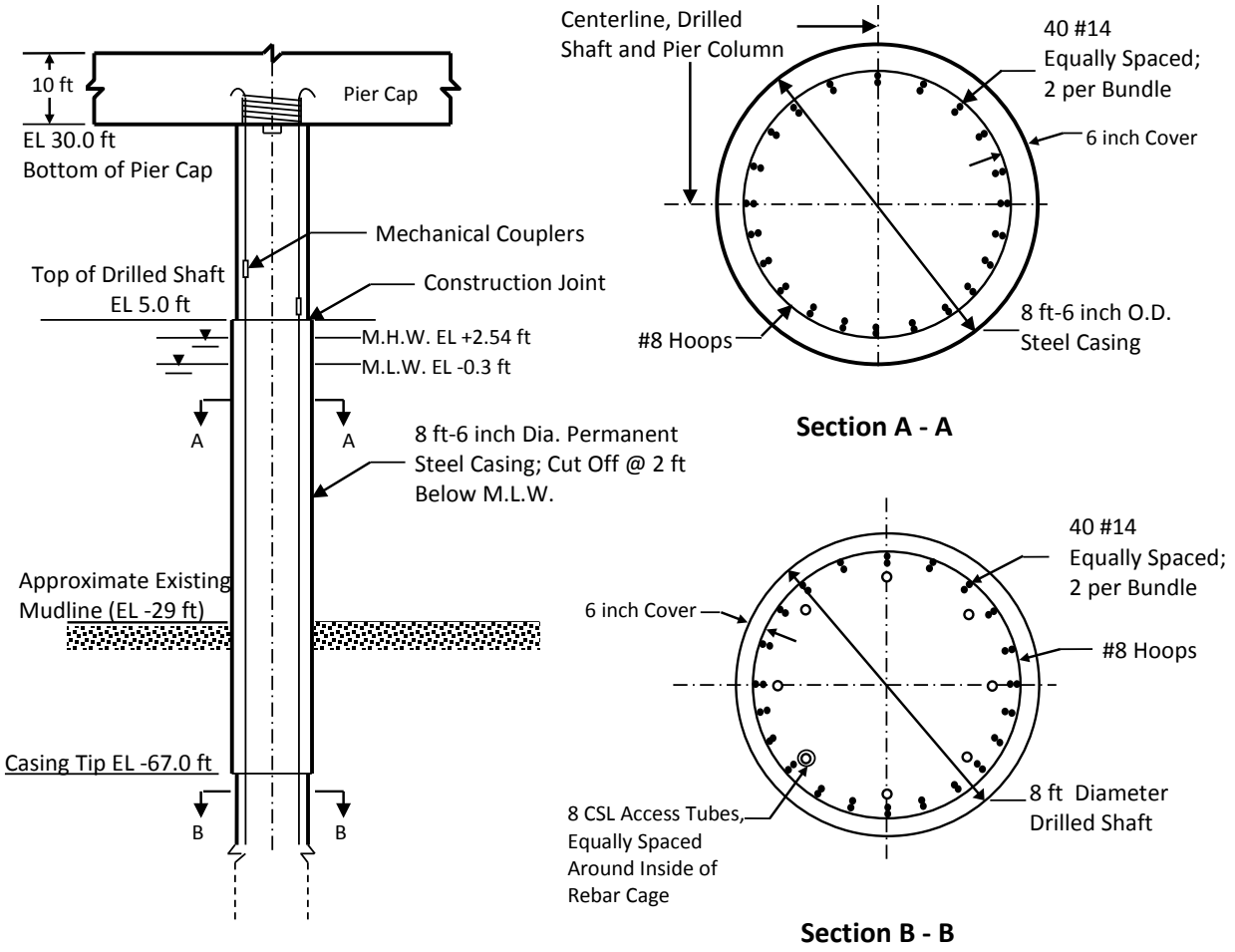


Figure A-12 Elevation and Cross-Sections of Drilled Shafts at Pier 2.

Step 13 Evaluate Constructability

Constructability issues associated with this project included: subsurface stratigraphy consisting of alternating layers of saturated coarse-grained and fined-grained soils; over-water construction; partial-length permanent casing; and requirements for concrete with high workability.

To address borehole stability, the contractor's installation plan called for the use of a polymer-based drilling fluid. Slurry properties were controlled in accordance with manufacturer-provided specifications. Prior to concrete placement, high silt content in the polymer slurry was addressed by full replacement of the slurry column with fresh slurry.

Drilled shafts were excavated from a barge using the crane-mounted rotary drilling machine shown in Figure A-13. Use of a crane mount provided the clearance needed for inserting and removing the drilling tool over the top of the permanent casing. Excavated soil was discharged from the auger into barges and transported to a disposal site. Excavated soil was not permitted to be discharged into the environmentally sensitive river environment or surrounding wetlands.



Figure A-13 Crane-Mounted Drilling Machine for Construction Over Water

The large volumes of concrete required made this project an excellent candidate for the use of high-performance concrete. The mix design selected corresponds to tremie concrete mixes described in Chapter 7 (Section 7.6.1.2) and was considered to self-consolidating concrete mix (SCC). The most significant advantages of using a tremie concrete SSC mix for this project included:

- High workability and retention of workability

  The volume of concrete required to construct 8-ft diameter, 201-ft long drilled shafts is approximately 360 cubic yards. This required 8 to 10 hours for placement of concrete, per drilled shaft. Retention of workability was achieved with an SCC mix design that included a high range water reducing admixture and a set retarder.

- Passing ability

  The tremie con mix was made with high sand content and No. 8 crushed stone, with a top-size of ½ inch. This mix provided adequate flow of concrete through the rebar cage, with a clear spacing of ten times the aggregate maximum particle size.

Table A-10 presents the mix design and properties of the concrete. This mix provided slump flow in the range of 18 to 24 inches and appeared to perform very well for this job. Figure A-14 shows a typical slump flow spread. No problems were observed during tremie placement of the concrete and no anomalies were detected by cross-hole sonic logging tests. Specifications called for a minimum 4,000 psi compressive strength, but this mix had 28 day breaks as high as 7,000 psi. Other noteworthy points regarding the concrete include:

- Class F fly ash was 20% of the total cementitious material
- Sand to total aggregate ratio = 0.48
- Water to cement ratio = 0.41
- The high-range water-reducing admixture was made from polycarboxylate polymers
- The hydration control admixture (retarder) provided some water reducing benefits

TABLE A-10 MIX DESIGN FOR DRILLED SHAFTS, DESIGN EXAMPLE

Mix Components

Mix Component	Pounds/Cubic Yard	Yield (Cubic Ft)
Type I Portland Cement	526	2.68
Class F Fly Ash	132	1.00
Concrete Sand	1363	8.30
No. 8 Crushed Stone	1500	8.71
Water	267	4.28
Total Air (%)	7.5±2.0	2.03
	Total	27.00

Admixtures

Admixtures	Ounces (US)/ Cubic Yard
High range water reducing and superplasticizing admixture meeting ASTM C-494 Types A and F	46.06
Water reducing and retarding admixture meeting ASTM T-494 Types B and D	26.32
Air entraining admixture meeting ASTM C-260	4.9

Mix Properties

Mix	Properties
Water to Cement Ratio	0.41
Concrete Unit Weight	140.3 (lb/ft^3)
Flow Spread of Fluid Concrete	19-24 inch



Figure A-14 Slump Flow Measurement on High-Performance Concrete of Design Example

A.3 SUMMARY

An example design problem, based on an actual bridge project, is presented to illustrate the design procedures presented in this manual. Design of single drilled shafts for one of the piers (Pier 2) is illustrated for Strength I, Service I, and Extreme Event I limit states. Information obtained from the subsurface investigation is used to establish an idealized subsurface profile and to select soil properties used in the design equations. Design scour conditions are incorporated into the Strength I and Service I limit state analyses. Calculations are presented to illustrate limit state checks for geotechnical and structural resistances, for lateral and axial loading. Finally, constructability issues relevant to the project are identified and the measures taken to address them are discussed.

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APPENDIX B

DRILLED SHAFTS IN CHALLENGING GEOMATERIALS

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APPENDIX B
DRILLED SHAFTS IN CHALLENGING GEOMATERIALS

Some ground conditions pose special challenges for design and/or construction for drilled shafts, either because of unusual geomaterial behavior or because of particular challenges with characterizing ground conditions and material properties for design. Experience has demonstrated that these special conditions may require methods adapted to the specific ground conditions present. Examples of several special geomaterials requiring special consideration are described in this appendix.

B.1 ARGILLACEOUS SEDIMENTARY ROCK

Fine-grained, argillaceous sedimentary rocks that include shale, claystone, siltstone, and mudstone are encountered over wide areas of the Central U.S. While some argillaceous rock can be quite competent and durable, these materials are often characterized as "weak rock" that can be difficult to effectively sample and test because it is often laminated and fissured. Some argillaceous rock formations can be effectively sampled by careful rock coring or using special samplers developed for hard soil or soft rock (e.g., Pitcher and Dennison samplers). However, recovered samples are often prone to rapid degradation when exposed such that it is often difficult to conduct common laboratory strength tests and field characterization tests like RQD. These difficulties introduce significant variability among laboratory and field test measurements, which in turn poses challenges for selecting appropriate input parameters for design. Special site characterization techniques and design methods have therefore been developed for design of drilled shafts in argillaceous rock formations. Because of the tendency for argillaceous rock to degrade upon exposure, these method generally rely on in situ tests that provide some measure of material strength and stiffness.

Several agencies use the so-called Texas Cone Penetration (TCP) Test for design of drilled shafts in argillaceous rock. The TCP test is a dynamic penetration test that is similar to the common Standard Penetration Test (SPT), but performed using a special 3-inch diameter solid steel cone instead of the Standard Sampler used for SPT measurements. Strictly speaking, the TCP test should be performed using a special 170 lb drop hammer with 24-inch drop height (Texas DOT, 2018). However, the test is sometimes performed using conventional SPT hammers (140-lb hammer and 30-inch drop height), which impart theoretical energies that are practically identical to that of the TCP hammer, in which case measurements should be reported as "modified" TCP measurements. In soft materials where the cone can be advanced 12 inches in 100 blows or less, the penetration resistance is reported as the number of blows required to penetrate the cone 12 inches (i.e., blows/ft), much like is done for conventional SPT measurements. However, in stiffer materials like most argillaceous rock formations, measurements are reported as the penetration distance measured for 100 hammer blows (typically in units of inches/100 blows).

The Texas, Oklahoma, and Missouri Departments of Transportation each have empirically developed methods for design of drilled shafts based on TCP or modified TCP measurements (Texas DOT, 2018; Pierce, 2013). Pierce (2013) found that empirical design methods based on TCP measurements involve considerable variability and uncertainty and require use of lesser resistance factors to achieve a given target reliability than design methods that rely on common laboratory test measurements. However, the methods are nevertheless useful and effective given that they are intended for use in conditions where conventional coring and laboratory testing is quite challenging. Some researchers have also developed empirical correlations between TCP measurements and properties of soft rock. For example, Cavusoglu et al. (2004) reports on correlations developed to relate TCP measurements to the compressive strength of

upper Cretaceous formation clay shales (from UU triaxial tests) and limestone (from uniaxial compression). The correlations are highly formation-dependent and exhibit a substantial scatter, but provide first order estimates of rock strength based on TCP measurements in formations where sample recovery is otherwise difficult.

The Colorado Department of Transportation developed a similar approach for design of drilled shafts in weak rock utilizing SPT measurements (Abu-Hejleh, et al., 2003). The standard penetration test is conducted in weak rock layers as long as the number of blows required to penetrate six inches is 50 or less (corresponding to $N < 100$). In such conditions, the field measured SPT N-values are used with empirically developed design equations to estimate side and tip resistances for drilled shafts (Abu-Hejleh et al. 2003; Abu-Hejleh and Atwoll, 2005; Abu-Hejleh, et al., 2005). When the number of blows to advance six inches exceeds 50, an attempt is made to use double-wall or triple-wall core barrels to obtain samples for core logging and laboratory strength testing that can be used for design. If adequate samples are not obtained, which is common when thinly bedded or highly jointed layers are encountered, pressuremeter testing (PMT) is used to estimate rock strength. Abu-Hejleh and Atwoll (2005) also provides empirical design equations for estimating load transfer curves for side and tip resistance as a function of SPT N-value for soil-like claystone and q_u for the harder claystone/sandstone that can be used to make simple estimates of axial load-settlement response of drilled shafts. Pierce (2013) describes evaluations performed to develop similar empirical methods based on SPT measurements in weak rock for use in Missouri.

B.2 LIMESTONE AND OTHER CARBONATE ROCKS

Geologic and engineering characteristics of carbonate rocks, including limestone and dolomite, vary widely depending upon geologic age, degree of weathering, and climate. Limestone formations range from hard and massive to weak, highly weathered, and karstic. For example, many limestone units encountered in midwestern states exhibit high compressive strengths and are relatively free of solution cavities, clay seams, and other problematic features. Design of drilled shafts in such materials can be performed using methods described for rock in the body of this manual. In contrast, limestone units in the southeastern U.S., and particularly Florida, are characterized by highly variable strength profiles, the presence of cavities which may be filled with soil, and interbedding of limestone with sand and marine clay layers (Crapps, 1986). Design of drilled shafts in these conditions requires special considerations. Lai (1998) describes design practice by the Florida DOT that is based on a modified version of a relationship by McVay et al. (1992) in which spatial variations in rock quality are incorporated by multiplying the unit side resistance by the average percent recovery (REC) of rock core expressed as a decimal. Lai (1998) also recommends using larger diameter double-tube core barrels (2.4-inch to 4-inch inner diameter) for obtaining samples of sufficient quality for laboratory strength tests.

Because of extreme variability and challenging site characterization, side resistance values in Florida limestone have also been evaluated using small-scale field pullout tests illustrated in Figure B-1 (Crapps, 1986). In such tests, a grout plug is formed in a 5.5-inch diameter cored hole at the bottom of a 6.5-inch diameter hole drilled to the test depth in rock. 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 2 ft in length and the average unit side resistance is taken as the measured pullout force divided by the sidewall interface area of the plug.

Brown (1990) describes design and construction challenges for drilled shafts in hard pinnacled limestones and dolomites encountered in the Valley and Ridge and Cumberland Plateau physiographic provinces of the southeastern U.S. Subsurface conditions are highly irregular due to extensive weathering. While

intact rock strengths may be high (up to 10,000 psi), numerous seams, slots, and cavities are typically filled with residual clayey soils as illustrated in Figure B-2. 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 prior to construction. Design, construction, and inspection must be flexible enough to adjust to conditions actually encountered. Use of probe holes for downhole inspection and identification of cavities and seams along the sides and beneath the tip is an essential part of the construction and inspection process (Figure B-3). To provide the flexibility needed for design, inspection, and construction, creative contracting approaches are also needed. Brown (1990) reports that contracting such work on a unit cost basis provides the flexibility needed to deal with unknown quantities of soil versus rock drilling, concrete overpours, rock anchoring, drilling of probe holes, etc. The engineer estimates unit quantities, but actual payment is based on unit costs of material quantities actually used. This requires careful inspection and record keeping.

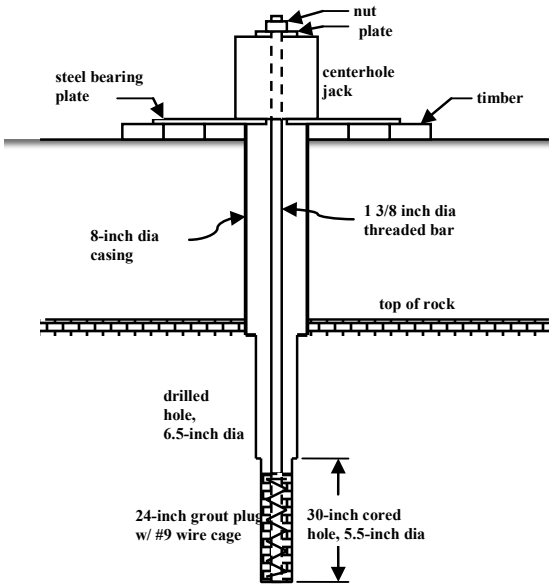


Figure B-1 Small-scale Pullout Test Used in Florida Limestone (after Crapps, 1986)

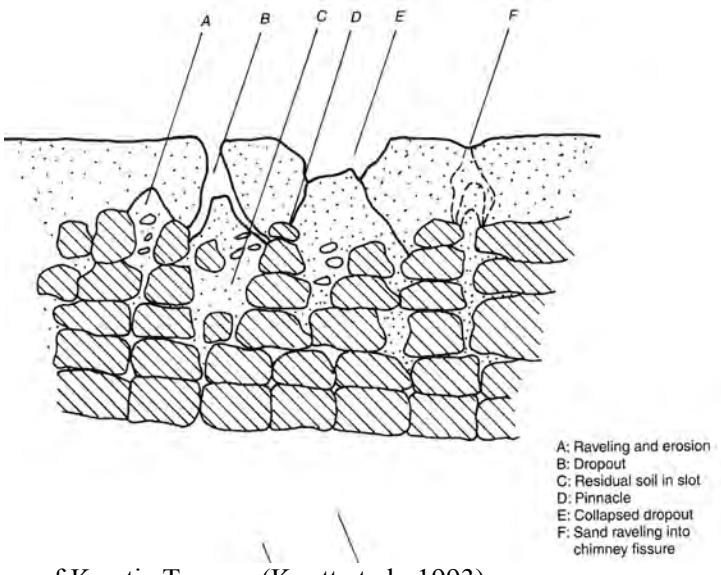


Figure B-2 Features of Karstic Terrane (Knott et al., 1993)

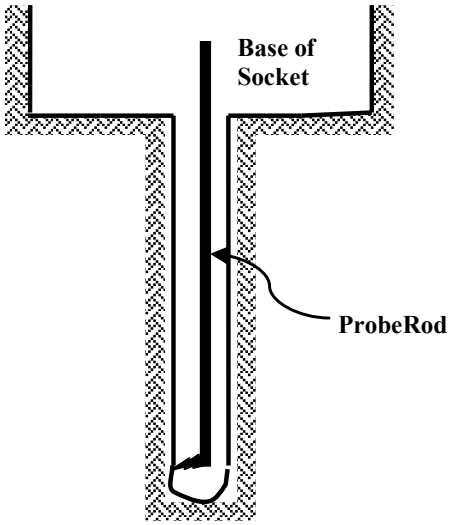


Figure B-3 Rock Probing Tool (after Brown, 1990)

B.3 GLACIAL TILL

The term till is applied to any unstratified and unsorted sediment carried or deposited directly by glacial ice. Till may be composed of particles of all sizes from clay up to cobbles and boulders, with the type of till varying from one glacier to another as well as within a single glacial environment. "Clay tills" are predominately composed of clay sized particles with varying amounts of sand, gravel, and larger particles, and are common in many parts of the upper Midwest. In contrast, "boulder tills" are composed for the most part of boulders and cobbles and are common in many parts of New England. Glacial deposits can be excellent bearing units for drilled shafts because of high strength and low compressibility, but characterizing engineering properties for design and construction can be a challenge due to sampling difficulties, limitations on in-situ testing, and in some cases highly complex subsurface conditions caused by multiple periods of glacial advance and retreat, weathering profiles, and interbedding of till with other glacial deposits. Flexible contracting practices suggested by Brown (1990) for highly variable ground conditions are therefore appropriate for use with many glacial till deposits.

Soliman (1983) describes a geotechnical investigation for foundation design in a dense, heavily overconsolidated glacial till of Wisconsin age on the southern shore of Lake Ontario in Sterling, New York. The till is described as silty to very silty, gravelly, fine to coarse sand (SM) with scattered cobbles and boulders and a trace of clay. Conventional sampling methods were unsuccessful due to the high density and high gravel and cobble content, a common occurrence in many till deposits that also makes use of SPT and CPT tests impractical. At the Sterling site, a Denison sampler equipped with a diamond bit and advanced by rotary drilling with downward pressure (similar to rock coring) resulted in samples adequate for laboratory testing. CD triaxial compression tests were used successfully to establish effective stress strength parameters (c' and ϕ') for foundation design. Pressuremeter tests (PMT) were used to establish values of modulus for settlement calculations. It was observed that modulus values from PMT were 2.6 to 6 times those obtained from triaxial tests. This result is consistent with results reported by other researchers (Klohn, 1965; Radhakrishna and Klym, 1974) who also found that in-situ modulus from PMT or field plate load tests ranges from 3 to 5 times greater than modulus from triaxial tests on cored samples of hard till. These experiences suggest that coring and use of PMT are useful site characterization tools in dense overconsolidated till deposits.

Lutenegger et al. (1983) describe how Wisconsin-age glacial till deposits in Iowa can be distinguished on the basis of geologic process into two categories of *basal till* and *diamictons* and how the properties of each differ in terms of texture, density, and structure. These properties in turn can be correlated to engineering properties including shear strength and compressibility. Figure B-4 shows the relationship between density and unconfined compressive strength for basal till deposits in eastern Iowa. According to the authors, a similar correlation was demonstrated for basal tills of similar texture (clay matrix) in Denmark, suggesting a universal application for basal tills of similar texture. The authors note that the Unified Soil Classification System is not useful for distinguishing between basal till and diamictons, as both materials typically are classified as CL, but that further analyses of grain size distributions show that diamictons are comprised of a wider range of particle sizes and are complexly interbedded with various types of stratified meltwater deposits. Recognizing these types of differences and understanding the geologic processes leading to formation of till deposits is key to formulating subsurface investigation strategies and understanding engineering properties.

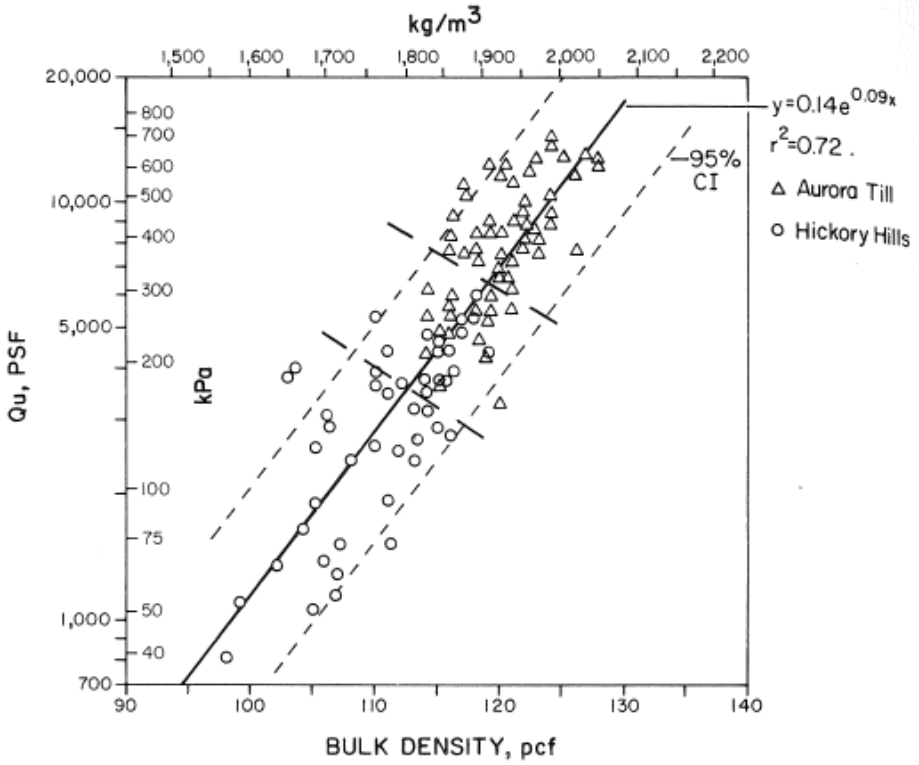


Figure B-4 Relationship Between Density and Unconfined Compressive Strength for Basal Tills in Eastern Iowa (Lutenegger et al., 1983).

B.4 PIEDMONT RESIDUAL SOILS

Geotechnical aspects of Piedmont residuum are described by several authors, including Martin (1977) and Sowers and Richardson (1983). Design and construction of drilled shafts in the Piedmont are described by Schwartz (1987), Gardner (1987), Mayne and Harris (1993), and Brown (2002). Mayne and Brown (2003) found that residual soils of the southern Piedmont are not particularly well-categorized by the Unified Soil Classification System (USCS). A vertical profile in the Piedmont may appear as if alternating strata of silty sands (SM) and sandy silts (ML) form the overburden. The strata seem to change in random fashion, suggesting high variability over short distances. This apparent variability is due to the fact that the mean grain size of the Piedmont residuum is close to the 75-micron criterion that separates

fine-grained from coarse-grained fractions (No. 200 sieve size). In fact, the Piedmont residuum acts more as a dual soil type (SM-ML), exhibiting characteristics of both fine-grained soils and coarse-grained soils.

Mayne et al. (2000) found that interpretation of in-situ tests, including SPT, CPT, and PMT, by conventional analyses can provide reasonable predictions of effective stress friction angle (ϕ') as measured in CU triaxial compression tests with pore pressure measurements. Use of SPT, CPT, and DMT can provide reasonable estimates of undrained shear strength, but users are referred to Mayne et al. (2000) for specific interpretation methods. The overall approach to estimating soil engineering properties is to obtain values of both drained and undrained strength parameters, analyze designs for both drained and undrained conditions, and base the final design on the critical loading mode. Calibration of design parameters determined from in-situ tests using laboratory test measurements is strongly recommended in these materials. CU triaxial tests with pore water pressure measurements offer the ability to establish strength and stress-strain properties for both drained and undrained conditions.

Other factors that make drilled shafts challenging to design and build in the Piedmont are:

- highly variable depths to bedrock
- presence of cobbles and boulders
- steeply dipping bedrock surfaces
- difficulties distinguishing between soil, partially weathered rock, and intact rock for pay purposes.

These factors make it difficult to determine final tip elevations for shafts required to reach intact rock prior to construction. Thorough site characterization and good communication between owners, engineers, and contractors are essential for developing strategies to deal with these issues. Schwartz (1987) describes sources of cost overruns and contractor-engineer-owner conflicts commonly encountered in drilled shaft projects in the Piedmont, and presents recommendations for addressing them.

B.5 CEMENTED SOILS

Cemented soils may be encountered in any location, but are widespread in the semi-arid and desert regions of the western U.S., in Florida, and along the banks of the lower Mississippi River. The effect of cementation on soil strength is to give the soil cohesion, c', which is otherwise not present in the uncemented soil. Strongly cemented soils can be advantageous for drilled shaft design and construction by providing high resistances and often providing an opportunity for the dry method of construction, eliminating the need for casing or slurry.

Cemented soils are a challenge for sampling, testing, and selection of appropriate engineering properties. If materials are weakly cemented, the cemented structure may not be recognized when conventional soil sampling techniques are used. SPT blow counts may appear to be uncharacteristically high, but recovered materials may appear to be uncemented due to destruction of the cementing bonds during drilling and sampling. Weak cementation may additionally be sufficient to preclude sampling using Shelby tubes. In both cases, the driving/pushing resistance may be mistakenly interpreted as being characteristic of dense uncemented sands. At the other extreme, strongly cemented soils may exhibit refusal when sampling with conventional soil sampling methods, requiring specialized samplers suitable for hard soil (Pitcher barrel or Dennison samplers) or rock coring.

Techniques to correctly identify cemented materials in borings include: (1) carefully observing recovered cuttings and attempting to recover small pieces of cementing materials that are returned with the

uncemented material; and (2) being observant of potential reasons for "anomalous" behavior during drilling and sampling (*e.g.*, uncharacteristically high blow counts during soil sampling, poor core recovery despite relatively uniform moderate drilling resistance, etc.). Observation of cemented soils in nearby roadcuts or other surface exposures may also indicate the presence of cemented deposits in the subsurface.

Caliche is a cemented geomaterial commonly encountered in the semi-arid and desert lands of the western U.S. While the term is loosely applied to any cemented soils, true caliche is considered to be the hard lithification of both fine-grained sediments and sand and gravel through secondary cementation by calcium and magnesium carbonate. Table B-1 summarizes nomenclature and drilling characteristics for caliche in the Las Vegas, NV area, where these deposits are widespread and important bearing units for both shallow and deep foundations (Cibor, 1983). Descriptions in Table B-1 illustrate the wide variety of material characteristics of cemented soils and suggests approaches for categorizing cemented soils and sampling strategies based on the categorization.

TABLE B-1 EXAMPLE CLASSIFICATION AND DRILLING/SAMPLING CHARACTERISTICS OF CALICHE, LAS VEGAS VALLEY (AFTER CIBOR, 1983)

Nomenclature		Hardness Classification	Drilling Rates[(1)] minutes/ft		Description of Material and Drill Cuttings
Cemented coarse-grained deposits	**Cemented fine-grained deposits**		**Without pulldown**	**With pulldown**	
Sand and gravel with scattered cementation	Decomposed caliche with silt and clay	Very still/very dense to slightly hard	-	-	Variable matrix of uncemented soil and cemented zones. Samples obtained with split-spoon or thick-walled sampler. Can be crumbled with fingers.
Partially cemented sand and gravel	Decomposed caliche	Moderately hard	< 5	< 3	Cemented to varying degrees. Fine-grained deposits sampled with thick-walled sampler; coarse-grained samples cannot be obtained with thick-walled sampler. Drilling produces large, rounded cuttings. Cuttings can be broken with difficulty with hands or easily when hammered.
Cemented sand and gravel	Weathered caliche	Hard	6 to 30	3 to 6	Visible chemical alterations from fresh deposits. Compressive strength similar to fresh deposits. Slight secondary porosity. Samples obtained by coring techniques. Drill cuttings less than ½ inch in diameter. Fragments can be broken with difficulty by hammering
	Fresh caliche	Very hard	700	70	No visible signs of chemical alteration. Non-porous. Resembles metamorphic or sedimentary rock. Drill cuttings less than 1/8 inchin diameter. Samples obtained by coring techniques. Fragments cannot be broken by hammering.

[(1)] using Mayhew 100 drill rig

Useful index tests on cemented soils include a simple unit weight determination on undisturbed specimens, which can help to determine whether the material is as dense as high blow counts might suggest. It may also be informative to either immerse specimens in water or add water to intact samples to assess whether the cementing agent is soluble or if the material softens when inundated with water. If either of these responses is identified, a careful assessment should be made of whether the service conditions will introduce water. If so, soil strength should be evaluated for the uncemented state.

Strongly cemented soils that can be sampled using coring techniques (e.g., caliche described as hard and very hard in Table B-1) can be treated as sedimentary rock for the purpose of foundation design. Uniaxial (unconfined) compressive strength should be measured in laboratory tests and design equations for nominal resistances given for rock can be applied to drilled shaft design. Load tests for drilled shafts in Las Vegas have showed unit side resistances in the range of 30 to 55 ksf in competent caliche layers, compared to 8 to 22 ksf in uncemented dense sand and gravel, and 4 ksf in stiff clay layers (Gura et al., 2007). No data are reported on measured strength of the caliche, but Cibor (1983) reports a range of 576 ksf to 1,440 ksf (4,000 to 10,000 psi) for compressive strength of competent caliche in the Las Vegas Valley, suggesting that methods presented in this manual for side resistance in rock apply to competent caliche. This hypothesis warrants further research.

For less cemented soils not exhibiting characteristics of rock, common laboratory strength testing is recommended. Because of the sensitive and brittle nature of the cementing materials, these tests must be carefully conducted and interpreted. It is important that representative samples be selected for laboratory testing, because a possible result of sample disturbance is that only the strongest materials may survive the drilling/sampling process. Block sampling has been shown to be an effective technique for obtaining samples of cemented sands suitable for laboratory testing. In situ testing using a pressuremeter has also been used effectively to provide quantitative strength and stiffness information. Other in situ testing techniques, specifically the SPT and dilatometer (DMT), provide useful qualitative results, but must be calibrated to specific site/material conditions to provide quantitative information.

The load-deformation response of cemented sands must also be reviewed from the perspective of the brittle character of the material. At low confining pressures, the response due to cementation will dominate the frictional response. This results in an initial stiff response due to the cementation, followed by a strain softening response associated with the frictional characteristics of the sand after rupture of cementing bonds. As confining pressures increase for a specific degree of cementation, the difference between the peak and post-peak strengths decreases. At high confining pressures, application of the confining pressure may disrupt cementing bonds, resulting in a load-deformation response that is consistent with that of an uncemented sand. The lesson from this general response characteristic is that the range of test confining pressures must be carefully selected to match the anticipated service conditions. Additionally, because cementing bonds can be disrupted at low strains, the anticipated strains under the anticipated working stress should be assessed to allow the engineer to decide whether peak (*i.e.*, cemented) or large-displacement (*i.e.*, uncemented) strengths should be used for design.

B.6 EXPANSIVE SOILS

Expansive geomaterials are those that exhibit large volume changes (shrinkage and swelling) in response to changes in water content. Fine-grained soils and sedimentary rock containing expansive clay minerals (*e.g.*, montmorillonite, compactions shales), and rock composed of crystalline hydrates such as gypsum are most likely to exhibit expansive behavior. Soils and rock exhibiting expansive behavior are common throughout much of the United States. Locations where expansive geomaterials are most prevalent include (1) Texas and along the Gulf Coast States, (2) the Appalachian states, (3) the Southwest, and (4) the Great Plains (Krohn and Slosson, 1980). Peck, et al. (1974) states that swelling soils are

"especially prevalent in a belt extending from Texas northward through Oklahoma, into the upper Missouri valley, and on through the western prairie provinces of Canada. In many parts of this belt, considerations of swelling dominate the design of foundations of structures." There are probably about twenty States in the United States where expansive clays present a problem to some degree (Gromko, 1974).

The first and most important step for design of drilled shafts in expansive soils is to determine whether the soil/rock at a given site is potentially expansive. A number of techniques are useful for identifying potentially expansive geomaterials, including:

- Observation of similar existing structures near the site,
- Identification of expansive clay minerals in surficial geomaterials (smectites, particularly those with sodium as the predominant exchangeable cation, are especially prone to swelling)
- Performance of swell pressure or volume-change tests (swell tests) using undisturbed specimens recovered from the site, and
- Use of published correlations with index properties of the soil.

Snethen et al. (1977) conducted a study for the U.S. Army Corps of Engineers to evaluate methods for identifying expansive geomaterials from common index properties. The resulting classification method is termed the "WES (Waterways Experiment Station) Classification Method" and is summarized in Table B-2. Explicit consideration should be given to the potential for expansive behavior during design for sites with surficial geomaterials that classify as having "high" swell potential. Explicit consideration of expansive behavior is not usually necessary for sites with surficial soil/rock that classifies as having "low" swell potential because normal design requirements are usually sufficient to overcome any limited effects of expansive soils. In "marginal" geomaterials, designers should rely on local experience with similar structures to decide whether to consider effects of expansive geomaterials explicitly.

TABLE B-2 METHOD OF IDENTIFYING POTENTIALLY EXPANSIVE SOILS (SNETHEN, ET AL., 1977)

Liquid Limit (%)	Plasticity Index	h (tsf) [a]	Potential Swell (%) [b]	Potential Swell Classification
< 50	< 25	< 1.5	< 0.5	low
50 – 60	25 - 35	1.5 – 4.0	0.5 – 1.5	marginal
> 60	> 35	> 4.0	> 1.5	high

[a] in situ total soil suction, which can be measured as prescribed in ASTM D 5298

[b] vertical swell at in situ overburden stress, which can be measured as prescribed in ASTM D 4546

Two fundamental issues must be considered when designing drilled shafts in expansive geomaterials. First, drilled shafts must be designed to extend below depths where substantial volume changes may occur so that foundations rest in stable materials that will limit the potential for volume changes to produce deformations within the supported structure. This is practically accomplished by requiring shafts to extend beyond depths where expansive geomaterials are present, or below the zone of seasonal moisture content changes if expansive geomaterials are present to great depths. Additionally, the shafts must be designed to resist uplift forces that may be imposed by swelling soils without compromising the performance of the drilled shafts. Loading induced by swelling soils is conceptually similar to downdrag loads induced by settling soils, except that swelling soils will impose upward force while downdrag

imposes downward force. Uplift forces are generally counteracted by loading from the superstructure, so uplift concerns are generally most significant for relatively lightly loaded foundations. Uplift is therefore seldom a concern for most bridges, but may be a concern for structures with more modest loading such as support facilities (e.g., structures for rest areas, toll plazas, agency support facilities) and foundations for signs and lighting. For such structures, drilled shafts must be design to have sufficient structural capacity to resist uplift forces, which may often require substantially greater reinforcing steel than would be needed to support other loads on the foundations.

Swelling is generally assumed to occur over depths where moisture contents change seasonally. No definitive method exists for establishing the zone of seasonal moisture change. The most effective means for establishing appropriate depths of seasonal moisture changes is from local/regional experience. Stroman (1986) recommends examining core samples from a soil boring to determine the depth to which the soil is jointed, perhaps slickensided, and blocky in structure. Color changes may also be evident at the bottom of the zone of seasonal moisture change. Stroman further notes that useful information on the depth of wetting and drying can be obtained by making extremely careful determinations of moisture content and by plotting these values as a function of depth. Water contents will frequently be more erratic nature in the zone of seasonal moisture content change. O'Neill and Poormoayed (1980) also describe a method wherein liquidity index values obtained from samples recovered over two or more seasons are plotted versus depth. The liquidity index will be rather scattered in the zone of seasonal moisture change but will approach a constant value within the zone of stable moisture.

Two general approaches can be adopted to address the potential for upward loading on drilled shafts due to swelling geomaterials, both of which are conceptually similar to approaches adopted for downdrag loading. The first approach is to isolate the drilled shafts within the “active zone” to avoid developing upward loading in the drilled shaft. This can be accomplished by using two casings within the “active zone”, with an annular space between the two casings. Kim and O'Neill (1996) describe use of two concentric lengths of pressed-fiber tubing separated by layers of asphalts of varying consistencies to isolate surficial expansive soil from drilled shafts. Raba (1977) also describes successful use of structural steel members surrounded by weak concrete as shown in Figure B-5. The second approach is to design the drilled shafts to have sufficient structural capacity to resist anticipated upward loads. A conservative approach is advised with this approach because of uncertainties regarding the magnitude of potential loads. The concept is similar to that used for designing drilled shafts for downdrag except that the shearing forces on the sides of a drilled shaft in the active zone are directed upward, usually counter to downward loading from the superstructure. For such analyses, it is generally prudent to assume that the full strength of the soil can be mobilized (i.e.,). It is also reasonable to assume that load and resistance factors for downdrag can be applied for such analyses.

A final point should be considered when constructing transportation facilities on sites with expansive soils. The underside of any structural members connecting drilled shafts near the final grade elevation, such as footings, caps, or beams, should be designed and constructed to have sufficient clearance above the ground surface to avoid uplift from swelling ground. A generous estimate of the amount of swell should be used for establishing the gap between the structure and ground surface.

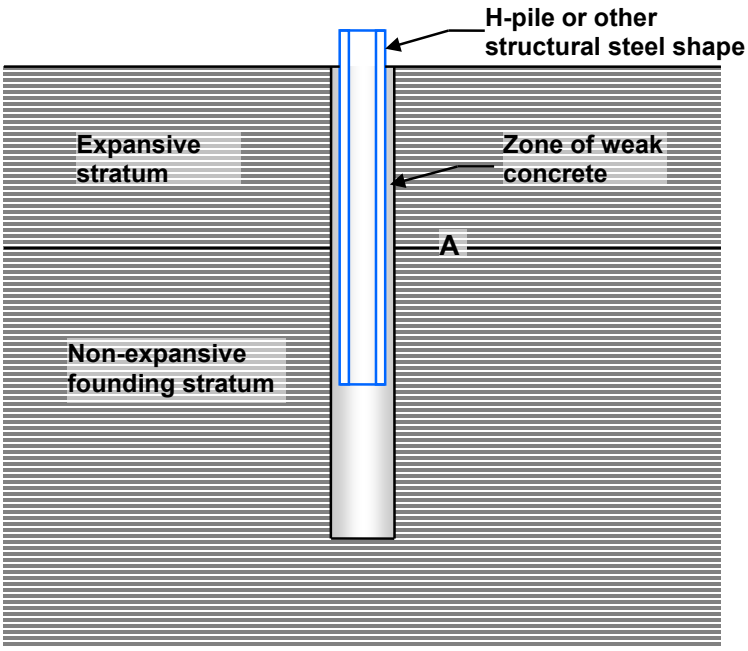


Figure B-5 Use of Embedded Structural Shape with Weak Concrete.

B.7 SUMMARY

The examples cited in this appendix represent geomaterials for which the drilled shaft engineering community has developed at least some level of experience. The list is by no means exhaustive and geotechnical engineers should always be attentive to materials that exhibit unusual behaviors due to their geologic origin, composition, or other factors not accounted for by conventional characterization methods. Unusual behavior might involve discrepancies between in-situ test results and expected behavior based on index properties, difficulty in obtaining representative samples, extremely high spatial variability in subsurface conditions, and expansive or collapsing behavior. For drilled shafts constructed in these conditions, or under any conditions that make it difficult to establish engineering properties of the supporting geomaterials, load testing and construction of technique shafts during the design phase should be considered. In most cases, the information obtained from technique shafts and design-phase load tests will yield more reliable designs and reduced risks of construction claims for differing site conditions and quantity overruns. Load test data from other nearby projects should also be utilized, whenever possible. If subsurface conditions can be shown to be similar, existing data can provide confidence in design parameters without the expense of a design-phase testing program.

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APPENDIX C

ANALYSIS OF AXIAL LOAD-DEFORMATION RESPONSE

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APPENDIX C
ANALYSIS OF AXIAL LOAD-DEFORMATION RESPONSE

Simple approximations of the axial load-deformation response of single drilled shafts under compression loading can be made on the basis of normalized displacement curves, such as Figure 10-10, developed by back-analysis of field load tests. Alternative approaches are available, some of which account more directly for soil-structure interaction and taking into account the soil and rock properties as well as characteristics of the shaft. These methods range from simple models based on elastic theory to sophisticated numerical methods requiring finite element computer codes. In this appendix, the essential features of methods that have proven useful for predicting the load-deformation response of drilled shafts for design applications are presented.

C.1 SIMPLE FORMULAS

This method provides an estimate of settlement for preliminary analysis or as an approximate check on other solutions. Vesic (1977) proposed the following equations to estimate settlement in the working load range of deep foundations in soils based on a general description of the soil and structural properties of the foundation.

$$w_T = w_c + w_{bb} + w_{bs} \qquad \text{C-1}$$

in which w_T = settlement of the head of the shaft, w_c = elastic compression of the reinforced concrete shaft, w_{bb} = settlement of the base due to load transferred to the base, and w_{bs} = settlement of the base due to load transferred along the sides. The term w_c can be approximated as:

$$w_c = \left(Q_h - 0.5\,Q_{ms}\right)\frac{L}{\left(AE\right)_{shaft}} \qquad \text{C-2}$$

in which L = length of the drilled shaft, A = cross-sectional area of the shaft, E = composite elastic modulus of the reinforced concrete shaft, Q_h = load applied to the head of the shaft, and Q_{ms} = mobilized side resistance. The effective elastic stiffness of the drilled shaft (product of elastic modulus and cross-sectional area) can be taken as:

$$(AE)_{shaft} = E_c\,(A_c + nA_s) \qquad \text{C-3}$$

where E_c = modulus of the concrete, A_c = cross-sectional area of concrete, A_s = cross-sectional area of longitudinal steel reinforcement, n = modulus ratio = E_s/E_c, and E_s = elastic modulus of steel reinforcement.

Vesic (1977) recommended the following expressions for w_{bb} and w_{bs}:

$$w_{bb} = C_p \left(\frac{Q_{mb}}{B\, q_{\max}} \right) \qquad \text{C-4}$$

$$w_{bs} = \left(0.93 + 0.16 \sqrt{\frac{L}{B}} \right) C_p \left(\frac{Q_{ms}}{L\, q_{\max}} \right) \qquad \text{C-5}$$

where Q_{mb} = load transferred to the shaft base, B = shaft diameter, L = shaft embedded length, and q_{max} = nominal unit base resistance (bearing capacity). C_p is a factor that depends on soil characteristics and can be approximated from Table C-1. Consistent units must be used in Equations C-1 through C-5.

TABLE C-1 VALUES OF C_P BASED ON GENERAL DESCRIPTION OF SOIL (VESIC 1977)

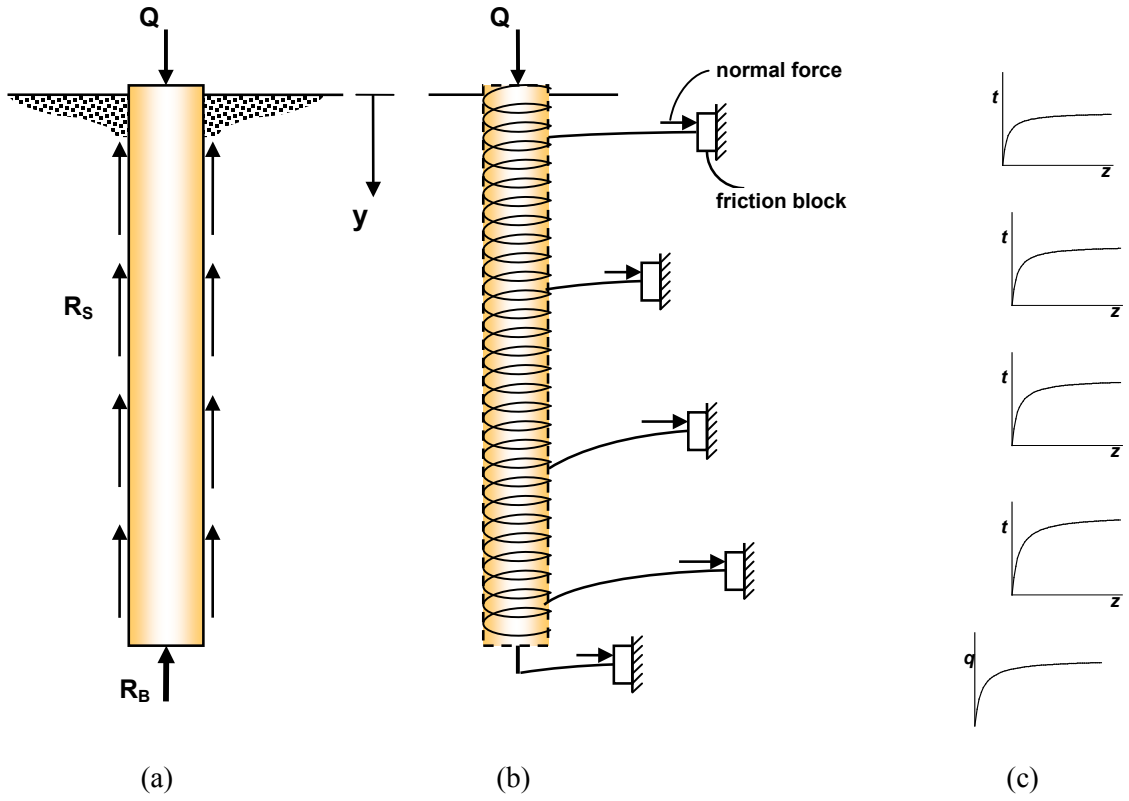
Soil Description	C_p
Sand (dense to loose)	0.09 – 0.18
Clay (stiff to soft)	0.03 – 0.06
Silt (dense to loose)	0.09 – 0.12

C.2 NUMERICAL SIMULATION OF LOAD TRANSFER

For layered soil profiles and/or where many potential loading cases and trial designs need to be analyzed, numerical simulation of the load-displacement curve is a practical alternative. Solutions are available in several formats including finite element analysis (FEM), boundary element analysis (BEM), and finite difference solutions. While FEM and BEM analyses may be justified on critical projects, the most widely used computer programs for this purpose employ a finite difference solution and this approach, known as the *t-z* curve method (Reese and Seed, 1957), is described.

A free body diagram of a drilled shaft is shown in Figure C-1a. The applied head load is resisted by the combined side (R_s) and base (R_B) resistances. In Figure C-1b, the drilled shaft and supporting soil are idealized as a system of springs. The single spring representing the concrete shaft is linear and is used to model the elastic stiffness of the shaft under an applied axial load. Soil-structure interaction along the sides and base of the shaft are represented by a series of mechanisms (nonlinear springs). One example of the type of mechanism that can be used is shown in Figure C-1b, and consists of a cantilever spring and a friction block.

Analytical functions are used to represent each of the interaction mechanisms. In Figure C-1c, a family of curves is shown to represent the nonlinear relationship between mobilized unit side resistance (*t*) and axial displacement (*z*) of the drilled shaft, referred to as a "side load-transfer function". A family of curves is used to illustrate that the *t-z* relationship can vary with depth.



(a) (b) (c)

Figure C-1 Model of Axially Loaded Shaft

The number of *t-z* curves used in a particular problem is determined by the need to capture the behavior of all the subsurface layers through which the shaft extends. The load-displacement response of the base of the shaft is modeled as a single nonlinear spring that describes the relationship between mobilized unit base resistance (q) and base displacement (w_b), referred to as a q-w_b curve or "base load-transfer function". Each nonlinear spring is discrete and independent of the other springs, meaning that this method does not satisfy continuity.

At any point along the shaft, the governing differential equation is given by:

$$EA\frac{d^2z}{dy^2} = \pi B t \qquad \text{C-6}$$

where EA = composite elastic axial stiffness of the reinforced concrete shaft (Equation C-3), B = shaft diameter, and depth is denoted by y. Solutions to Equation C-6 require a mathematical function relating the two variables t and z (or q and w at the base). In general, the load transfer functions can be expressed by:

$$t = \psi\,(z, y)\,z \qquad \text{C-7}$$

where ψ denotes the secant slope to the *t-z* curve at a specified value of deflection z_y, as shown in Figure C-2. Substituting Equation C-7 into Equation C-6 yields:

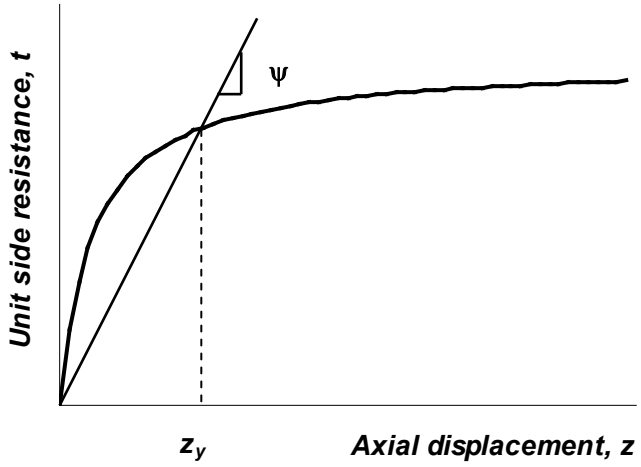


Figure C-2 Nonlinear *t-z* Relationship

$$\frac{d^2z}{dy^2} - \left(\frac{\pi B}{EA}\right)\psi(z,y)z = 0 \qquad \text{C-8}$$

Equation C-8 can be solved numerically using a finite difference scheme with a specified boundary condition (load applied to the top of the shaft). For each value of axial load, the numerical solution yields values of axial displacement (z) and mobilized unit side resistance (t) for each specified value of depth (y). From these data, a load versus head displacement curve can be constructed for the drilled shaft. Also, for each load increment, a curve of load versus depth (load transfer curve) can be calculated and constructed (*e.g.*, Figure 10-1c). A detailed description of the finite difference equations used in the *t-z* curve method is beyond the scope of this manual. However, several commercial software programs are available to solve this problem and the program documentation can be consulted for detailed information on the numerical methods.

The degree to which the *t-z* curve method models the actual behavior of a drilled shaft depends upon the *t-z* and q-w_b curves (functions) used in the analysis. Load transfer functions can be estimated in a variety of ways, including finite element studies, elasto-plastic modeling of the soil-shaft interface, load test measurements, and laboratory shear tests. Commercially available software programs that perform this type of analysis provide the user with the option of using built-in load transfer functions or specifying user-generated curves. The built-in functions are based on recommendations developed from full-scale load tests in a variety of soil types. The most widely-used and cited *t*-z and q-w_b curves for drilled shafts are those presented in the 1999 version of this manual (O'Neill and Reese, 1999) and reproduced below in Figure C-3. User-generated curves can also be developed from laboratory triaxial or direct shear tests in which the stress-strain behavior of the soil or soil-concrete interface is measured (for example, see Kraft et al., 1981; Stanton et al., 2015). Some researchers have developed recommendations for drilled shaft *t-z* curves in specific geomaterials, for example, cemented calcareous fine-grained soils (Walsh et al., 1995), Florida limestone (Niraula, 2004), and others. However, research has not yet advanced to the point where load transfer curves can be predicted for all conditions with confidence. Construction practices and the particular response of a given soil or rock to drilling and concrete placement may also affect side and base load transfer.

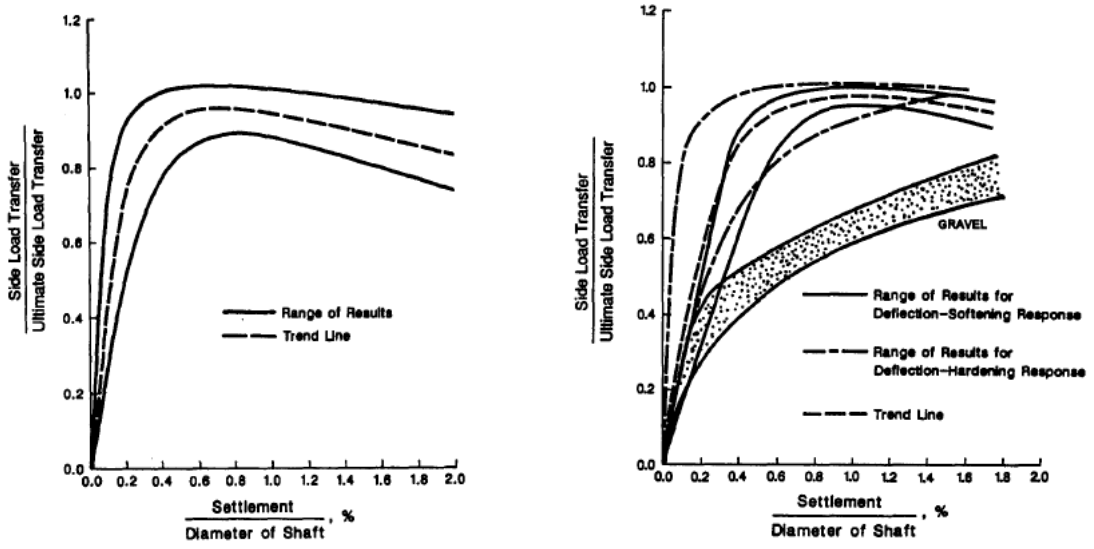


Figure C-3 Normalized side load transfer curves for (a) cohesive and (b) cohesionless soils

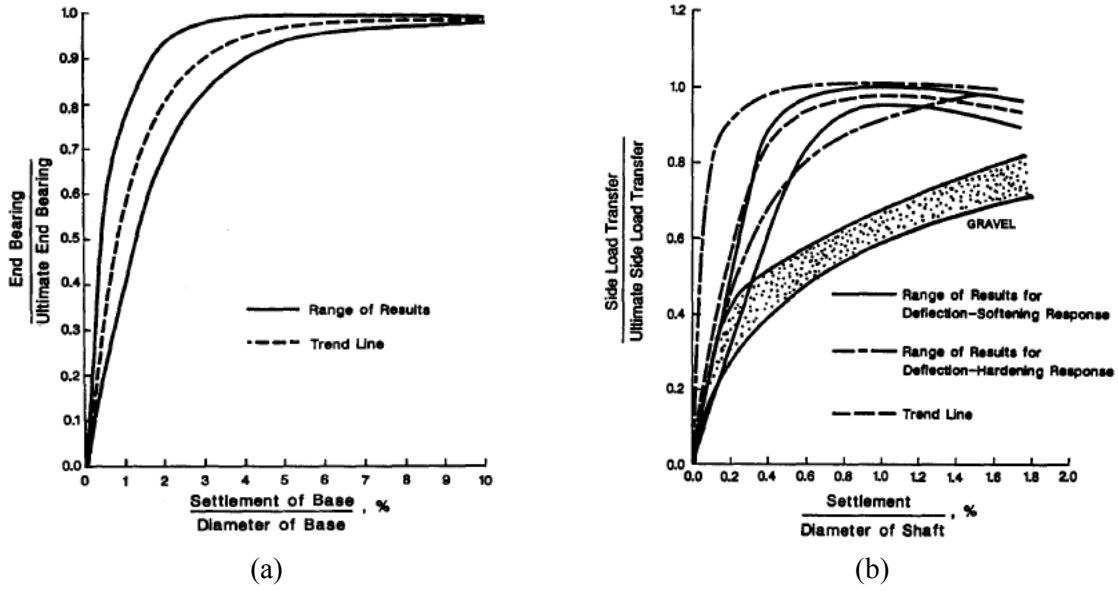


Figure C-4 Normalized base load transfer curves for (a) cohesive) and (b) cohesionless soils

An effective way to utilize the *t-z* curve method is in conjunction with field load tests. This approach is particularly useful on projects where the subsurface stratigraphy is complex and load testing is conducted with instrumentation needed to establish the load transfer in each soil or rock layer. The numerical solution can be calibrated to establish agreement with load transfer determined from the load test, then utilized to model the load-deformation response of trial designs that may involve different depths and diameters than those of the load test.

C.3 APPROXIMATE CLOSED-FORM SOLUTIONS

Closed-form solutions based on simplifying assumptions about the load transfer behavior of axially loaded drilled shafts can be valuable design aids. To be valid, closed form solutions should be verified by reasonable agreement with more sophisticated finite element analyses and by agreement with field load tests. The value of closed-form solutions lies in their ease of application, for example requiring a set of calculations that can be accomplished with a spreadsheet, thus enabling the designer to assess quickly the influence of design variables on the settlement behavior of the shaft. All such models are limited to the conditions assumed in their derivation, and care must be exercised not to apply the model to conditions outside of the validated range. Two such methods are described below, both of which are based on a procedure developed by Randolph and Wroth (1978) for elastic analysis of piles. The general form of the equations is therefore similar in both methods, but with different assumptions about the distribution of elastic modulus with depth.

C.3.1 Rock Sockets (Kulhawy and Carter 1992)

An approximate method given by Kulhawy and Carter (1992) provides simple closed-form expressions that compare reasonably well to more sophisticated nonlinear finite element analyses reported by Pells and Turner (1979) and Rowe and Armitage (1987). The basic problem is depicted in Figure C-5a 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 ν_c. The rock mass surrounding the cylinder is homogeneous with Young's modulus E_r and Poisson's ratio ν_r while the rock mass beneath the base of the shaft has Young's modulus E_b and Poisson's ratio ν_b. The solution (Figure C-5b) approximates the load-deformation response of an axially loaded rock socket as consisting of two linear segments: (1) the initial linear elastic response and (2) the full slip condition. The maximum load is limited to the nominal axial resistance.

For compression loading, two cases are treated by Kulhawy and Carter (1992): (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. The shear socket solution would be useful in analyzing a load test in which base resistance is eliminated by creating a void beneath the base of the drilled shaft. Only the complete socket case will be treated here.

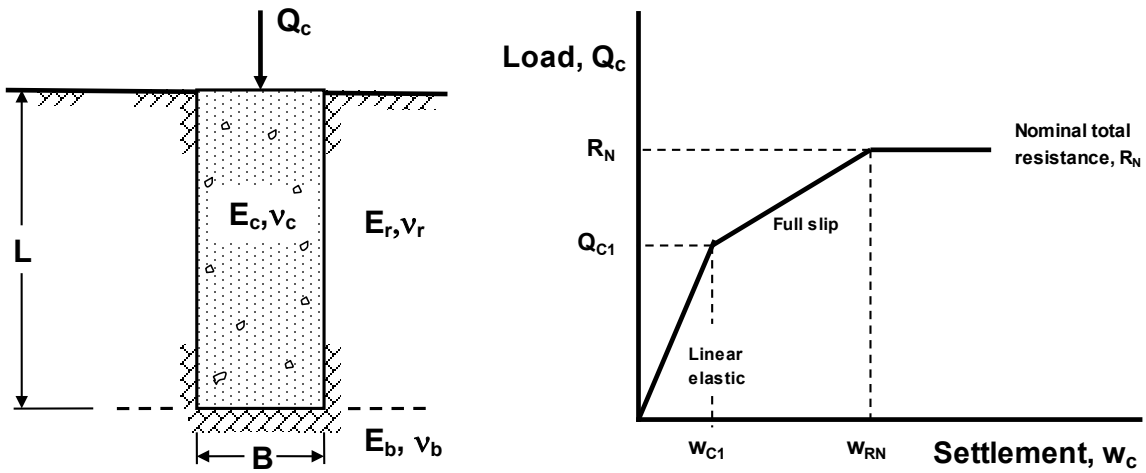


Figure C-5 Simplified Model of Axial Load-Deformation Behavior, Drilled Shaft in Rock; (a) Elastic model of Rock Socket; (b) Computed Bilinear Load-settlement Curve

1. For the linearly elastic portion of the load-displacement curve.

$$w_c = \frac{2Q_c}{G_r B} \frac{1+\left(\frac{4}{1-\nu_b}\right)\left(\frac{1}{\pi\lambda\xi}\right)\left(\frac{2L}{B}\right)\left(\frac{\tanh[\mu L]}{\mu L}\right)}{\left(\frac{4}{1-\nu_b}\right)\left(\frac{1}{\xi}\right)+\left(\frac{2\pi}{\zeta}\right)\left(\frac{2L}{B}\right)\left(\frac{\tanh[\mu L]}{\mu L}\right)} \quad \text{C-9}$$

in which:

$$(\mu L)^2 = \left(\frac{2}{\zeta\lambda}\right)\left(\frac{2L}{B}\right)^2 \quad \text{C-10}$$

$$\zeta = \ln [5(1 - \nu_r)L/B] \quad \text{C-11}$$

$$\lambda = E_c/G_r \quad \text{C-12}$$

$$G_r = E_r/ [2(1 + \nu_r] \text{ = elastic shear modulus of rock mass} \quad \text{C-13}$$

$$\xi = G_r/G_b \quad \text{C-14}$$

$$G_b = E_b/ [2(1 + \nu_b] \quad \text{C-15}$$

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

$$\frac{Q_b}{Q_c} = \frac{\left(\frac{4}{1-\nu_b}\right)\left(\frac{1}{\xi}\right)\left(\frac{1}{\cosh[\mu D]}\right)}{\left(\frac{4}{1-\nu_b}\right)\left(\frac{1}{\xi}\right)+\left(\frac{2\pi}{\zeta}\right)\left(\frac{2D}{B}\right)\left(\frac{\tanh[\mu D]}{\mu D}\right)} \quad \text{C-16}$$

2. For the full slip portion of the load–displacement curve.

$$w_c = F_3\left(\frac{Q_c}{\pi E_r B}\right) - F_4 B \quad \text{C-17}$$

in which:

$$F_3 = a_1(\lambda_1 BC_3 - \lambda_2 BC_4) - 4a_3 \quad \text{C-18}$$

$$F_4 = \left[1 - a_1 \left(\frac{\lambda_1 - \lambda_2}{D_4 - D_3}\right) B\right] a_2 \left(\frac{c}{E_r}\right) \quad \text{C-19}$$

$$C_{3,4} = \frac{D_{3,4}}{D_4 - D_3} \quad \text{C-20}$$

$$D_{3,4} = \left[\pi\left(1-\nu_b^2\right)\left(\frac{E_r}{E_b}\right) + 4a_3 + a_1\lambda_{2,1}B\right]\exp\left[\lambda_{2,1}D\right] \quad \text{C-21}$$

$$\lambda_{1,2} = \frac{-\beta \pm \left(\beta^2 + 4\alpha\right)^{1/2}}{2\alpha} \quad \text{C-22}$$

$$\alpha = a_1\left(\frac{E_c}{E_r}\right)\left(\frac{B^2}{4}\right) \quad \text{C-23}$$

$$\beta = a_3\left(\frac{E_c}{E_r}\right)B \quad \text{C-24}$$

$$a_1 = (1 + \nu_r)\zeta + a_2 \quad \text{C-25}$$

$$a_2 = \left[\left(1 - \nu_c\right)\left(\frac{E_r}{E_c}\right) + \left(1 + \nu_r\right) \right]\left(\frac{1}{2\tan\phi\tan\psi}\right) \quad \text{C-26}$$

$$a_3 = \left(\frac{\nu_c}{2\tan\psi}\right)\left(\frac{E_r}{E_c}\right) \quad \text{C-27}$$

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

$$\frac{Q_b}{Q_c} = P_3 + P_4\left(\frac{\pi B^2 c}{Q_c}\right) \quad \text{C-28}$$

in which:

$$P_3 = a_1(\lambda_1 - \lambda_2)\, B \exp[(\lambda_1 + \lambda_2)D] / (D_4 - D_3) \quad \text{C-29}$$

$$P_4 = a_2(\exp[\lambda_2 D] - \exp[\lambda_1 D]) / (D_4 - D_3) \quad \text{C-30}$$

Note that the point of intersection between the linear elastic portion of the curve and the full slip segment, defined by point (Q_{C1}, w_{C1}) in Figure C-3b, can be calculated by setting Equation C-9 equal to Equation C-17, solving for the resulting value of axial load (Q_{C1}) and using this value to compute the corresponding displacement w_{C1}.

Numerical solutions to Equations C-9 through C-30 are implemented easily by spreadsheet or other desktop computational tools, thus providing designers a simple analytical tool for assessing the likely ranges of behavior for trial designs. The user of this method should be familiar with the assumptions made in its development. The modulus of the rock mass is assumed to be constant over the depth of shaft embedment and beneath the base. Rock mass modulus and its variation with depth must, therefore, be assessed carefully and determined to satisfy the assumption of uniformity. Strength of the rock mass is required in terms of its Mohr-Coulomb parameters (c, ϕ, and ψ) where ψ = angle of dilatancy. In the absence of laboratory testing of the rock-concrete interface, for example by constant normal stiffness direct shear tests, Kulhawy and Carter (1992) suggest the following correlations between the Mohr-Coulomb strength parameters and uniaxial compressive strength (q_u) of intact rock:

$$\frac{c}{p_a} = 0.1\left(\frac{q_u}{p_a}\right)^{2/3} \quad \text{C-31}$$

$$\tan\phi\tan\psi = 0.001\left(\frac{q_u}{p_a}\right)^{2/3} \quad \text{C-32}$$

To illustrate the application of this method, consider Illustrative Example 19-4 Chapter 10) in which a 2-ft diameter, 7.5-ft deep socket in competent limestone was analyzed for nominal axial resistance. The mean uniaxial compressive strength was 870 psi and the calculated total compressive resistance (side plus base) was 1,751 kips. Figure C-6 shows a spreadsheet solution in which the load-displacement behavior of the trial shaft is predicted and plotted using Equations C-9 through C-32.

The above method is best applied in conjunction with load testing of rock sockets, providing a framework for interpretation of the load test results by establishing values of the rock mass properties (E_r, E_b, c, ϕ, and ψ). Where borings verify that the rock mass has similar lithology, strength, and discontinuity characteristics, the analysis can then be used to evaluate load-deformation behavior of trial designs. When combined with appropriate judgment and experience, the above approach represents a reasonable analysis of drilled shaft rock sockets.

C.3.2 Dense Residual Piedmont Soils (Mayne and Harris 1993)

The following method suggested by Mayne and Harris (1993) also is based on the elasticity solution developed by Randolph and Wroth (1978). The authors showed reasonably good agreement between the elastic analysis and the load-displacement response of drilled shafts in residual Piedmont soils as measured in field load tests. The method also showed good agreement with results of load tests on drilled shafts in very dense granular glacial till (O'Neill et al., 1996) and was recommended for dense cohesionless soils with N-values greater than 50. The method requires calculation of nominal unit side and base resistances (f_{SN}, q_{BN}) using the methods presented in Chapter 10 (β-method for side resistance; Equation 10-14 for base resistance).

The following correlation equation is recommended to establish values of the soil modulus (E_s) from Standard Penetration Test results:

$$\frac{E_s}{p_a} = 22\left(N_{60}\right)^{0.82} \qquad \text{C-33}$$

in which: p_a = atmospheric pressure in the same units as E_s, and N_{60} = SPT N-value corrected for 60% energy efficiency. A best-fit straight line is used to model the distribution of soil modulus with depth, as illustrated in Figure C-7. The following modulus values are then defined: E_{sL} = value of soil modulus at the base of the shaft for the undisturbed in situ condition; E_b = effective modulus at the base of the shaft accounting for stress relief due to excavation, taken as 0.4 E_{sL}, and E_{sm} = value of modulus at mid-depth of the shaft (L/2).

The load versus settlement curve is then modeled as being bilinear, as illustrated in Figure C-5b, with a limiting value corresponding to the nominal axial compressive resistance, R_N. The endpoints of the linear elastic and full slip segments are determined by the origin and the two points defined by coordinates (Q_{C1}, w_{C1}) and (R_N, w_{RN}). At the upper end of the linear elastic segment, the full side resistance and some portion of the base resistance have been developed. At the end of the full slip segment, the full side and base resistances have been developed. The computations used to establish the points on the curve are as follows:

Elastic Load-Settlement Analysis, Rock Socket in Compression
***by* Kulhawy & Carter (1992)**

SHAFT PROPERTIES		PROPERTIES OF SIDE ROCK MASS		PROPERTIES OF BASE ROCK MASS	
Shaft diameter B (in):	72	Rock mass modulus E_r (k/in^2):	450	Rock mass modulus E_b (k/in^2):	450
Shaft depth D (in):	120	Rock mass Poissons ratio, ν_r:	0.25	Rock mass Poissons ratio, ν_b:	0.25
Modulus of concrete, E_c (kip/in^2):	5075	Shear modulus, G_r (k/in^2):	180	Shear modulus, G_b (k/in^2):	180
Poissons ratio, ν_c:	0.25	cohesion, c (lb/in^2)	45.2		
Shear modulus, G_c (kip/in^2):	2030	friction angle, ϕ (degrees)	41		
		dilation angle, ψ (degrees)	2		

DIMENSIONLESS CONSTANTS FOR LINEAR ELASTIC (*calculated*)	
$\lambda = E_c/G_r$	28.19444444
$\zeta = \ln[5(1-\nu_r)\,D/B]$	1.832581464
$\xi = G_r/G_b$	1
$(\mu D)^2$	0.430091298
μ	0.005465112
for complete socket: $G_r B w_c / 2Q_c$	0.076561479
for shear socket: $E_r B w_c / 2Q_c$	0.249244998

Equivalent Linear Spring Constants K

complete socket: $K_c = Q_c/w_c$	84637.86292	k/in
shear socket: $K_s = Q_c/w_c$	64996.3	k/in

Proportion of Load Transferred to Tip

complete socket: $Q_{tip}/2Q_c$	0.283880186

CALCULATED CONSTANTS FOR FULL SLIP	
a_3	0.3174
a_2	21.6843
a_1	23.9750
β	257.7263
α	350418.6
λ_1	0.0014
λ_2	-0.0021
C_1	-1.9445
C_2	-2.9445
C_3	0.0637
C_4	1.0637
D_3	0.4632
D_4	7.7291
F_1	13.9560
F_2	0.0022
F_3	2.7300
F_4	0.0004
P_3	0.752071669
P_4	-1.193356254

Linear Elastic Load-Displacement Calculations

Q_c (kips)	*Complete Socket* w_c (in)
0	0.00
2100	0.02

Full-Slip Load-Displacement Calculations

2100	0.03
30649	0.79
30649	1.90

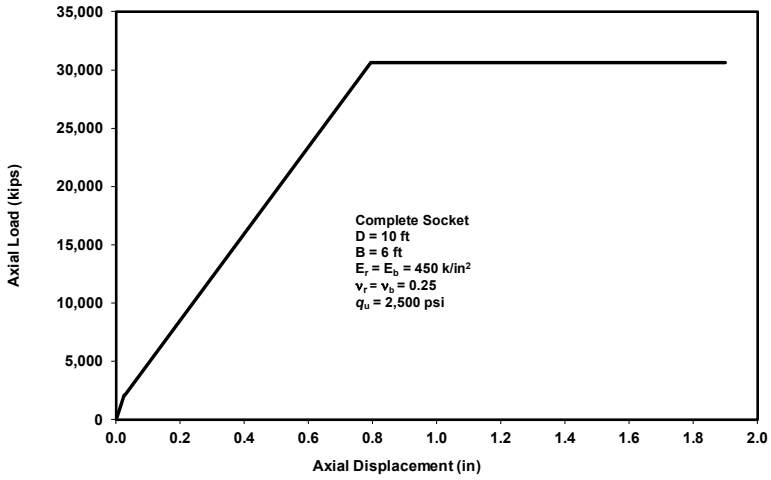


Figure C-6 Spreadsheet Analysis to Evaluate Load-Displacement of Rock Socket as Described in Illustrative Example 10-4

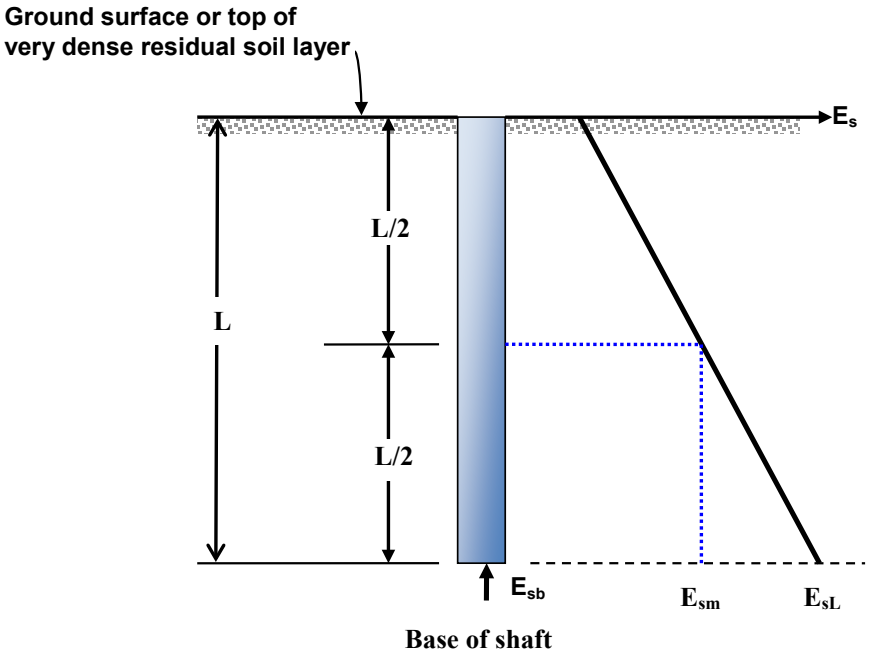


Figure C-7 Variation of Soil Modulus with Depth

$$Q_{C1} = \frac{f_{SN}\,\pi\,BL}{1-\left\{\dfrac{I}{\left[\xi\cosh(\mu L)\right]\left(1-\nu^2\right)}\right\}} \qquad \text{C-34}$$

where I = elastic influence factor given by:

$$I = 4(1+\nu)\left\{\frac{1+\dfrac{8\tanh(\mu L)L}{\pi\,\lambda(1-\nu)\xi(\mu L)L}}{\dfrac{4}{(1-\nu)\xi}+\dfrac{4\pi\dfrac{E_{sm}}{E_{sL}}\tanh(\mu L)L}{\zeta\,(\mu L)B}}\right\} \qquad \text{C-35}$$

$$\mu L = \left(\frac{L}{B}\right)2\sqrt{\frac{2}{\zeta\,\lambda}} \qquad \text{C-36}$$

$$\zeta = \ln\left\{\left[0.25+\left(2.5\left(\frac{E_{sm}}{E_{sL}}\right)(1-\nu)-0.25\,\xi\right)\right]\frac{2L}{B}\right\} \qquad \text{C-37}$$

$$\lambda = 2(1+\nu)\frac{E_c}{E_{sL}} \qquad \text{C-38}$$

$$\xi = \frac{E_{sL}}{E_b} = 2.5 \qquad \text{C-39}$$

in which:

ν= Poisson's ratio of the soil
E_c = elastic modulus of the reinforced concrete shaft (composite)

The displacement corresponding to load Q_{C1} is given by:

$$w_{C1} = \frac{Q_{C1}\, I}{E_{sL}\, B} \qquad \text{C-40}$$

The maximum load is the summation of the nominal side and base resistances, or;

$$R_N = f_{SN}(\pi BL) + q_{BN}\left(\frac{\pi B^2}{4}\right) \qquad \text{C-41}$$

The additional displacement that occurs over the full slip segment is computed by considering that any additional load beyond Q_{C1} is transferred to the base, yielding the following:

$$\Delta w = (Q_T - Q_{T1})\frac{(1-\nu^2)}{E_b\, B} \qquad \text{C-42}$$

The above method has been shown to give reasonable results for diameters up to 5 ft and is also implemented conveniently using a spreadsheet to perform the calculations.

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APPENDIX D

GUIDE DRILLED SHAFT CONSTRUCTION SPECIFICATION

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APPENDIX D
GUIDE DRILLED SHAFT CONSTRUCTION SPECIFICATION

SECTION X - DRILLED SHAFTS

X.1 DESCRIPTION

This item of work shall consist of furnishing all materials, labor, tools, equipment, services and incidentals necessary to construct the drilled shafts in accordance with the Contract Documents and this Specification.

X.2 SUBMITTALS, APPROVALS AND MEETINGS

At least four weeks prior to the start of drilled shaft construction, the Contractor shall submit four copies of a project reference list to the Engineer for approval, verifying the successful completion by the Contractor of at least three separate foundation projects within the last five years with drilled shafts of similar size (diameter and depth) and difficulty to those shown in the Plans, and with similar subsurface geotechnical conditions. A brief description of each project and the owner's contact person's name and current phone number shall be included for each project listed.

COMMENTARY

Electronic versions of all submittals should be encouraged. All submissions should be made concurrently to all on the distribution list.

X.2.1 Experience and Personnel

At least two weeks prior to the start of drilled shaft construction, the Contractor shall submit four copies of a list identifying the on-site supervisors and drill rig operators assigned to the project to the Engineer for approval. The list shall contain a detailed summary of each individual's experience in drilled shaft excavation operations, and placement of assembled reinforcing cages and concrete in drilled shafts.

- On-site supervisors shall have a minimum of two years experience in supervising construction of drilled shaft foundations of similar size (diameter and depth) and difficulty to those shown in the Plans, and similar geotechnical conditions to those described in the geotechnical report. The work experience shall be direct supervisory responsibility for the on-site drilled shaft construction operations. Project management level positions indirectly supervising on-site drilled shaft construction operations *will* not be acceptable for this experience requirement.
- Drill rig operators shall have a minimum one year experience in construction of drilled shaft foundations.

The Engineer *will* approve or reject the Contractor's qualifications and field personnel within ten working days after receipt of the submission. Work shall not be started on any drilled shaft until the Contractor's qualifications and field personnel are approved by the Engineer. The Engineer may suspend the drilled shaft construction if the Contractor substitutes unapproved field personnel without prior approval by the Engineer. The Contractor shall be fully liable for the additional costs resulting from the suspension of work and no adjustments in contract time resulting from such suspension of work will be allowed.

X.2.2 Drilled Shaft Installation Plan

At least four weeks prior to the start of drilled shaft construction, the Contractor shall submit four copies of a drilled shaft installation plan narrative for acceptance by the Engineer. In preparing the narrative, the Contractor shall reference the available subsurface geotechnical data provided in the Contract boring logs and any geotechnical report(s) prepared for this project. This narrative shall provide at a minimum the following information:

- Description of overall construction operation sequence and the sequence of drilled shaft construction when in groups or lines.
- A list, description and capacities of proposed equipment, including but not limited to cranes, drills, augers, bailing buckets, final cleaning equipment and drilling unit. As appropriate, the narrative shall describe why the equipment was selected and describe equipment suitability to the anticipated site and subsurface conditions. The narrative shall include a project history of the drilling equipment demonstrating the successful use of the equipment on shafts of equal or greater size in similar subsurface geotechnical conditions.
- Details of drilled shaft excavation methods, including proposed drilling methods, methods for cleanout of the bottom of the excavation hole, and a disposal plan for excavated material and drilling slurry, and regulated/hazardous waste (if applicable). If appropriate this shall include a review of method suitability to the anticipated site and subsurface geotechnical conditions, including boulder and obstruction removal techniques if such are indicated in the Contract subsurface geotechnical information *or Contract Documents*.

COMMENTARY

A drilled shaft installation plan is an essential document that should be required for all projects that require drilled shaft foundations. The Plan should be specific to the particular project and not just a standard list of general steps typical of drilled shaft construction. The Plan should fully document the equipment and procedures to be used in the work, allowing the Engineer opportunity in advance of construction to confirm compliance of the Contractor's proposed equipment and procedures with the Contract Documents, and to identify proposed practices that may adversely influence the nominal resistance or serviceability characteristics of the completed shafts. A drilled shaft installation plan also effectively communicates to shaft construction personnel and field inspectors the project requirements for drilled shaft installation. The Plan also serves as a basis for identifying any unapproved modifications to the equipment and procedures used for installation and testing of the drilled shafts.

The Contracting Agency/Designer should review the list of Plan items identified herein, and supplement the list, as appropriate, for each project.

The *Contracting Agency/Designer* should recognize that the depth of the requested narrative should be appropriate to the complexity of the project.

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, insert the following: "Variations in the bearing layer elevation from that shown in the Plans are anticipated. The Contractor shall have equipment on-site capable of excavating an additional 20% of depth below that shown in the Plans."

- Details of the method(s) to be used to ensure drilled shaft hole stability (i.e., prevention of caving, bottom heave, etc. using temporary casing, slurry, or other means) during excavation and concrete placement. The details shall include a review of method suitability to the anticipated site and subsurface geotechnical conditions.

  COMMENTARY

  Where the installation of drilled shafts will take place adjacent to existing sensitive installations prone to damage due to the instability of uncased drilled shaft holes or where subsurface soil strata do not lend themselves to an uncased construction technique due to stability concerns, the owner may specify the use and limits of the temporary casing.

  In areas of the country that are subject to high seismic forces, the designer may limit the drilled shaft diameter to that used in design calculations. Such limitations may include restrictions on telescoping of casing or limiting the amount of excavation prior to the introduction of casing.

- Detailed procedures for mixing, using, maintaining, and disposing of the slurry shall be provided. A detailed mix design (including all additives and their specific purpose in the slurry mix), and a discussion of its suitability to the anticipated subsurface geotechnical conditions shall also be provided for the proposed slurry.
- The submittal shall include a detailed plan for quality control of the selected slurry, including tests to be performed, test methods to be used, and minimum and/or maximum property requirements which must be met to ensure that the slurry functions as intended, considering the anticipated subsurface conditions and shaft construction methods, in accordance with the slurry manufacturer's recommendations and these Specifications.

As a minimum, the slurry quality control plan shall include the following tests:

Property	Test Method
Density (lb/ft^3)	Mud Weight (Density), API 13B-1, Section 1
Viscosity (seconds/quart)	Marsh Funnel and Cup, API 13B-1, Section 2.2
pH	Glass Electrode, pH Meter, or pH Paper
Sand Content (%)	Sand, API 13B-1, Section 5

- Reinforcing steel shop drawings, details of reinforcement placement including type and location of all splices, reinforcement cage support and centralization methods, *type and location of all spacers, cross-hole sonic logging tubes and other instrumentation, and procedures for lifting and setting the reinforcement cage.*
- Where casings are proposed or required, casing dimensions and detailed procedures for permanent casing installation, temporary casing installation and removal, and methods of advancing the casing, along with the means to be utilized for excavating the drilled shaft hole in accordance with Articles X.4.1 through X.4.3.
- Where temporary casing is used, details of the method to extract the temporary casing and maintaining shaft reinforcement in proper alignment and location, and maintaining the concrete slump to keep concrete workable during casing extraction.

- Details of concrete placement, including a time schedule, proposed *equipment and procedures for delivering concrete to the drilled shaft, placement of the concrete into the shaft,* operational procedures for pumping, and a sample uniform yield form to be used by the Contractor for plotting the volume of concrete placed versus the depth of shaft for all shaft concrete placement.
- Procedures for tremie methods used, *including initial placement and the raising of the tremie or pump line during placement, and size of tremie and pump lines*.

  COMMENTARY

  Anomalies due to head loss of tremie need to be addressed as to the procedure to avoid, inspect and repair, if needed.

- The method to be used to form a horizontal construction joint during concrete placement.

  COMMENTARY

  Horizontal construction joints *are undesirable and should generally be avoided, but* may become necessary due to equipment breakdown or loss of concrete supply during drilled shaft concrete placement.

  Where top of drilled shafts cutoff elevations are below the water surface, a sealed cofferdam arrangement is generally required to construct the joint.

- Where applicable, a description of the material to be used to temporarily backfill a drilled shaft excavation hole during a stoppage of the excavation operation, as well as the method used to place and remove the material.
- *Details of procedures to prevent loss of slurry or concrete into waterways, sewers and other areas to be protected*.
- A description of the method and materials that will be used to fill or eliminate all voids below the top of shaft between the plan shaft diameter and excavated shaft diameter, or between the shaft casing and surrounding soil, if permanent casing is specified.

  COMMENTARY

  In seismic design situations, the backfill material and placement method should attempt to replicate the existing ground conditions as closely as possible.

- Methods of removal and disposal of contaminated concrete.
- *Details of any required load tests including equipment, instrumentation, procedures, calibration data for test equipment, calculations, and drawings.*
- *Details and procedures for protecting existing structures, utilities, roadways and other facilities during drilled shaft installation.*
- *Other information required by the Plans or specified herein.*

The Engineer *will* evaluate the *drilled* shaft installation plan for conformance with the Contract Plans and Specifications within ten working days after receipt of the submission. At the option of the *Contracting Agency*, a Shaft Installation Plan Submittal Meeting may be scheduled following review of the Contractor's initial submittal of the Plan. Those attending the Shaft Installation Plan Submittal Meeting shall include the following:

- The superintendent, on-site supervisors, and other Contractor personnel involved in the preparation and execution of the drilled shaft installation plan.
- The Project Engineer and *Contracting Agency's* personnel involved with the structural, geotechnical, and construction review of the *drilled* shaft installation plan together with *Contracting Agency's* personnel who will provide inspection and oversight during the drilled shaft construction phase of project.

The Contractor shall submit any significant updates or modifications to the drilled shaft installation plan whenever such updates or modifications are proposed to the Engineer. The Engineer will evaluate the new information for conformance with the Contract Plans and Specifications within ten working days after receipt of the submission.

X.2.3. Slurry Technical Assistance

If slurry is used to construct the drilled shafts, the Contractor shall provide, or arrange for, technical assistance from the slurry manufacturer as specified in Subsection X.4.3.4.1. The Contractor shall submit four copies of the following to the Engineer:

- The name and current phone number of the slurry manufacturer's technical representative assigned to the project.
- The name(s) of the Contractor's personnel assigned to the project and trained by the slurry manufacturer's technical representative in the proper use of the slurry. The submittal shall include a signed training certification letter from the slurry manufacturer for each individual, including the date of the training.

X.2.4 Approvals

Work shall not begin until all the required submittals have been accepted in writing by the Engineer. All procedural acceptances given by the Engineer *will* be subject to trial in the field and shall not relieve the Contractor of the responsibility to satisfactorily complete the work.

X.2.5 Drilled Shaft Preconstruction Conference

A shaft preconstruction conference shall be held at least five working days prior to the Contractor beginning any shaft construction work at the site to discuss investigative boring information, construction procedures, personnel, and equipment to be used, and other elements of the accepted *drilled* shaft installation plan as specified in Article X.2.2. If slurry is used to construct the shafts, the frequency of scheduled site visits to the project site by the slurry manufacturer's representative shall be discussed. Those attending shall include:

- The superintendent, on site supervisors, and other key personnel identified by the Contractor as being in charge of excavating the shaft, placing the casing and slurry as applicable, placing the steel reinforcing bars, and placing the concrete. If slurry is used to construct the shafts, the slurry manufacturer's representative and a Contractor's employee trained in the use of the slurry, as identified to the Engineer in accordance with Article X.4.3.4.1, shall also attend.

  COMMENTARY

  Meetings may need to be held in order to obtain agreement on the shaft submittal, but the shaft conference should only be held after approval.

- The Project Engineer, key inspection personnel, and appropriate representatives of the Owner.

COMMENTARY

Attendees on *Contracting Agency's* behalf should include representatives having experience in construction, materials, *and* structural and geotechnical design.

If the Contractor's key personnel change, or if the Contractor proposes a significant revision of the approved *drilled* shaft installation plan, an additional conference may be held at the request of the Engineer before any additional shaft construction operations are performed.

X.2.6 *Logs of Shaft Construction*

The Contractor's Quality Control staff shall prepare inspection logs documenting each shaft construction activity, including casing installation, excavation, shaft bottom inspection, reinforcement installation and concrete placement. The logs shall fully document the work performed with frequent reference to the date, time and casing/excavation elevation. In addition, the Contractor shall prepare and submit the logs documenting any subsurface investigation borings or rock core holes performed for the Contract at drilled shaft foundation locations.

Records for temporary and permanent casing shall include at least the following information: identification number and location of the shaft; diameter and wall thickness of the casing; dimensions of any casing reinforcement; top and bottom elevations of the casing; method and equipment used for casing installation; any problems encountered during casing installation; and the name of the inspector.

COMMENTARY

Complete and detailed shaft construction records are particularly important since these records are used to confirm compliance with the design and specification requirements, are often used for determination of pay quantities and, most importantly, they provide an initial means for identifying potential defects in the shaft, as well as their cause and approximate location.

It is generally preferable that the Engineer or other representative of the Contracting Agency be given responsibility for performing shaft construction inspection. In such cases, the requirements for logging shaft construction may be omitted from the Specification. However, for design-build projects it is often required that drilled shaft inspection be performed by an independent QC entity engaged by the Contractor. In such cases, the specification provisions included herein would be applicable.

The shaft excavation log shall contain at least the following information: identification number, location and surface elevation of the shaft; description and approximate top and bottom elevation of each soil or rock material encountered; seepage or groundwater conditions; type and dimensions of tools and equipment used, and any changes to the tools and equipment; type of drilling fluid used, if any, and the results of slurry tests; any problems encountered; elevation of any changes in the shaft diameter; method used for bottom cleaning; final bottom elevation of the shaft; and the name of the inspector and the date, time and name of any changes in the inspector.

COMMENTARY

If the Contractor is given the responsibility for logging shaft construction operations, the Specifications should include a copy of the forms to be used. Many Contracting Agencies have standard forms for their projects. Sample inspection forms are also available from the FHWA (FHWA, 2002) and are presented in Appendix E.

Concrete placement records shall include at least the following information: concrete mix used; time of start and end of concrete placement; volume and start/end time for each truck load placed; concrete test results; concrete surface elevation and corresponding tremie tip elevation periodically during concrete placement; concrete yield curve (volume versus concrete elevation, actual and theoretical); and the name of the inspector.

The logs for each shaft construction activity shall be submitted to the Engineer within 24 hours of the completion of that activity. A full set of shaft inspection logs for an individual drilled shaft shall be submitted to the Engineer within 48 hours of the completion of concrete placement at the shaft.

X.3 MATERIALS

X.3.1 Concrete

Concrete used in the construction of drilled shafts shall conform to Article _____, "Classes of Concrete." The concrete slump shall be as follows:

Dry placement methods:	6.0 – 8.0 in.
Casing removal methods:	8.0 – 10.0 in.
Tremie placement methods:	8-0 – 10.0 in.

Slump loss of more than 4 in. shall not be permitted during the period equal to the anticipated pour period plus 2.0 *hours*. A minimum of 6.0 in. *slump* shall be required for this time period. Slump life may be extended through the use of retarders and mid-range water reducers, if approved by the Engineer.

COMMENTARY

This article should refer to the appropriate section of the Specification that addresses concrete material requirements. Each Contracting Agency will likely have their own concrete mix designs for drilled shaft applications based on local practice, weather conditions, aggregates, etc. Desirable properties for a concrete mix used for drilled shaft applications are: fluidity, compaction, resistance to segregation, limited bleed water, controlled heat of hydration, controlled set time, and minimum required strength.

To achieve maximum workability, the following mix characteristics are recommended:

- A maximum course aggregate *size* of 0.375 in. in wet hole pours or shafts with dense reinforcing configurations.
- Use of rounded in lieu of crushed aggregates.
- Consider using fly ash as a cement replacement and as a fluidifier. *Use of fly ash as a cement replacement can also help mitigate heat of hydration concerns.*

In cases where dense reinforcing configurations close to the minimum opening size limits are specified it is suggested that slumps of 8.0 – 10.0 in. be used even in dry placement methods.

X.3.2 Reinforcing Steel

Reinforcing steel used in the construction of *drilled* shafts shall conform to AASHTO M31M/M31.

When necessary, reinforcing steel shall be bundled in order to meet the clear spacing requirements between the vertical reinforcement bars. Rolled hoops or bundled spirals shall be used in order to maximize the clear space between horizontal reinforcement.

COMMENTARY

Current practice regarding minimum clear space between reinforcement elements is to have clear distance between parallel longitudinal and parallel transverse reinforcing bars not be less than five times the maximum aggregate size or 5.0 in., whichever is greater, per Article 5.12.9.5.2 of the AASHTO LRFD Bridge Design Specifications. Recent research indicates that clear distance between parallel longitudinal and parallel transverse reinforcing bars of ten times the maximum aggregate size provides for improved flow of concrete through the cage to ensure the integrity of the concrete outside of the reinforcing cage. Prevailing practice varies regarding minimum opening size amongst various *Contracting Agencies*. Their experience indicates that the current requirements contained in AASHTO LRFD Bridge Design Specifications produce desired results when the requirements in the construction specifications are fully applied.

Reinforcing steel for shafts poured inside temporary casings should not have hooks to the outside.

For projects where the tip elevation of the drilled shafts depends on a minimum penetration into a bearing stratum, the length of the drilled shaft and the corresponding length of the reinforcing cage cannot be determined until the top of the bearing stratum is identified during shaft excavation. To avoid delay in the fabrication of the reinforcing cage in such cases, and the resulting delay in the completion of shaft construction, a provision can be included in the specifications requiring extension of at least one-half of the longitudinal bars, as well as all of the spiral or hoop steel once the final length of the shaft is determined. The increased length in the drilled shaft and shaft reinforcement can be based on the increased quantities for these work items. However, payment is not made for additional reinforcement if the shaft is inadvertently overexcavated to a depth greater than shown on the Plans or required by the Engineer.

X.3.3 Casings

All permanent structural casing shall be of steel conforming to ASTM A 36/A36M or ASTM 252 Gr 2 unless specified otherwise in the Plans. All splicing of permanent structural casing shall be in accordance with Article 6.13.3, "Welded Connections," of the AASHTO LRFD Bridge Design Specifications.

The diameter of permanent casing shall be as shown on the Plans, unless a larger diameter casing is approved by the Engineer. When a larger size permanent casing is approved by the Engineer, no additional payment will be made for the increased weight of casing steel, or the increased quantity of drilled shaft excavation and concrete.

All permanent casing shall be of ample strength to resist damage and deformation from transportation and handling, installation stresses, and all pressures and forces acting on the casing. For permanent nonstructural casing, corrugated casing may be used.

COMMENTARY

Permanent structural casing is defined as casing designed as part of the shaft structure, providing stiffness or load carrying capacity, and installed to remain in place after construction is complete.

All temporary casing shall be a smooth wall structure steel, except where corrugated metal pipe is shown in the Plans as an acceptable alternative material. All temporary casing shall be of ample strength to resist damage and deformation from transportation and handling, installation and extraction stresses, and all pressures and forces acting on the casing. The casing shall be capable of being removed without deforming and causing damage to the completed shaft, and without disturbing the surrounding soil.

COMMENTARY

Temporary casing is defined as casing installed to facilitate shaft construction only; is not designed as part of the shaft structure, and is completely removed after shaft construction is complete, unless otherwise shown in the Plans *or approved by the Engineer*. *Examples of temporary casing that may be required to be left in place include casing for drilled shaft installed through a body of water or through very soft soils that may not provide adequate lateral support for the wet concrete.*

Permanent nonstructural casing is defined as casing designed to remain in place to assist in the construction of the drilled shaft.

The outside diameter of *temporary* casing shall not be less than the specified diameter of the *drilled* shaft.

The casing shall be watertight and clean prior to placement in the excavation.

Where seismic design requires that the shaft be constructed to the diameters indicated in the *Plans*; telescoping casing *shall not be used*.

COMMENTARY

In cases where seismic design governs, the inside diameter of the *temporary* casing shall not be greater than the specified diameter of the *drilled* shaft plus 6.0 in., unless otherwise specified on the Plans *or approved by the Engineer*.

In cases where seismic design governs, the Engineer may need to re-evaluate the foundation performance with the added stiffness of the casing with wall thickness greater than that shown on the Plans.

Where the minimum thickness of the casing is specified in the Plans, it shall be considered to satisfy structural design requirements only. The Contractor shall increase the casing thickness from the minimum specified thickness, as necessary, to satisfy the construction installation requirements.

Temporary casing shall be completely removed, unless otherwise shown on the Plans or approved by the Engineer.

X.3.4. Mineral Slurry

Mineral Slurry shall be used in conformance with the quality control plan specified in Article X.2.2.

COMMENTARY

Unit weights stated are exclusive of weighting agents that may be proposed by the Contractor with the agreement of the slurry manufacturer's representative.

Mineral slurry shall conform to the following requirements:

Property	Test	Requirement
Density (lb/ft^3)	Mud Weight (Density) API 13B-1, Section 1	64.3 to 72
Viscosity (seconds/quart)	Marsh Funnel and Cup API 13B-1, Section 2.2	28 to 50
pH	Glass Electrode, pH Meter, or pH Paper	8 to 11
Sand Content (%) (immediately prior to placing concrete)	Sand, API 13B-1, Section 5	4.0 max.

COMMENTARY

Some slurry systems incorporate a weighting agent when utilizing salt water in slurry. This may add up to 5 lb/ft^3 to the unit weight.

Where it is necessary to use a mineral slurry in salt water applications, it is recommended that attapulgite or sepiolite be used in lieu of bentonite.

When approved by the Engineer, slurry may be used in salt water, and the allowable densities may be increased up to 2 lb/ft^3. Slurry temperature shall be at least 40ºF when tested.

X.3.5. Polymer Slurry

Polymer slurries shall be used in conformance with the manufacturer's recommendations, and shall conform to the quality control plan specified in Article X.2.2. Only synthetic slurry systems which have been approved by the *Contracting Agency* shall be used. The polymer slurry *shall* conform to the following requirements:

Property	Test	Requirement
Density (lb/ft^3)	Mud Weight (Density) API 13B-1, Section 1	64 max.
Viscosity (seconds/quart)	Marsh Funnel and Cup API 13B-1, Section 2.2	32 to 135
pH	Glass Electrode, pH Meter, or pH Paper	8 to 11.5
Sand Content (%) (immediately prior to placing concrete)	Sand, API 13B-1, Section 5	1.0 max.

COMMENTARY

The range of properties specified in the table is typical for many of the polymer slurries made from anionic polyacrylamides, or PAMs, on the market at the present time (2018). However, other varieties of polymer products are available. Accordingly, adjustments may be necessary and appropriate based on recommendations provided by the polymer supplier or manufacturer, taking into account new products and job-specific conditions. For example, some of the proprietary polymers now on the market operate optimally at Marsh funnel viscosities up to 150.

When approved by the Engineer, polymer slurry may be used in salt water, and the allowable densities may be increased up to 2 lb/ft^3.

The sand content of polymer slurry prior to final cleaning and immediately prior to placing concrete shall be less than or equal to 1.0 percent, in accordance with American Petroleum Institute API 13B-1, Section 5.

Slurry temperature shall be at least 40°F when tested.

X.3.6. Water Slurry

Water may be used as slurry when casing is used for the entire length of the drilled hole, *provided that the method of drilled shaft installation maintains stability at the bottom of the shaft excavation.*

COMMENTARY

A water slurry is water that is maintained as clean a possible during its use. The mixing of water with naturally occurring site materials is not recommended.

Water slurry shall conform to the following requirements:

Property	Test	Requirement
Density (lb/ft^3)	Mud Weight (Density) API 13B-1, Section 1	64 max.
Sand Content (%)	Sand, API 13B-1, Section 5	1.0 max.

When approved by the Engineer, slurry may be used in salt water, and the allowable densities may be increased up to 2 lb/ft3.

Slurry temperature shall be at least 40°F when tested.

X.3.7 Access Tubes for Cross-Hole Sonic Log Testing

Access tubes for cross-hole sonic log testing shall be steel pipe of 0.145 in. minimum wall thickness and at least 1.5 in. inside diameter.

The access tubes shall have a round, regular inside diameter free of defects and obstructions, including all pipe joints, in order to permit the free, unobstructed passage of 1.3 in. maximum diameter source and receiver probes used for the cross-hole sonic log tests. The access tubes shall be watertight, free from corrosion, with clean internal and external faces to ensure good bond between the concrete and the access tubes. The access tubes shall be fitted with watertight threaded caps on the bottom and the top.

COMMENTARY

PVC has been used for cross-hole sonic logging (CSL) access tubes, but PVC tubing has a greater likelihood of debonding from the concrete, and associated influence on the CSL test results. PVC tubing also is more prone to damage during lifting and setting of the reinforcement cage. Accordingly, steel pipe is the preferred material for CSL access tubes.

X.3.8 Grout

Grout for filling the access tubes at the completion of the cross-hole sonic log tests shall be a neat cement grout with a minimum water/cement ratio of 0.45.

X.4 CONSTRUCTION

X.4.1 Drilled Shaft Excavation

Drilled shafts shall be excavated to the required depth as shown in the Plans or as directed by the Engineer. Once the excavation operation has been started, the excavation shall be conducted in a continuous operation until the excavation of the shaft is completed, except for pauses and stops as noted, using approved equipment capable of excavating through the type of material expected. Pauses during this excavation operation, except for casing splicing and removal of obstructions, *will* not be allowed. The Contractor shall provide temporary casing at the site in sufficient quantities to meet the needs of the anticipated construction method.

COMMENTARY

The equipment and procedures used for drilled shaft excavation should be consistent with design assumptions regarding load transfer from the drilled shaft to the surrounding ground. For example, pre-drilling or the use of permanent non-structural casing within the bearing stratum that may result in reduced side resistance may not be appropriate for drilled shafts that rely on side resistance unless the influence of these procedures has been addressed by the Engineer in the determination of shaft penetration requirements.

Pauses, defined as interruptions of the excavation operation, may be allowed only for casing splicing and removal of obstructions. *Drilled* shaft excavation operation interruptions not conforming to this definition shall be considered stops.

If the *drilled* shaft excavation is not complete at the end of the shift or series of continuous shifts, the shaft excavation operation may be stopped, provided the Contractor, before the end of the work day, protects the shaft as indicated in Article X.4.2.

If slurry is present in the shaft excavation, the Contractor shall conform to the requirements of Article X.4.3.4.2 regarding the maintenance of the minimum level of drilling slurry throughout the stoppage of the shaft excavation operation, and shall recondition the slurry to the required slurry properties in accordance with Article X.3 prior to recommencing shaft excavation operations.

The excavation and drilling equipment shall have adequate capacity, including power, torque and down thrust to excavate a hole of both the maximum diameter and to a depth of 20 ft, or 20 percent, beyond the maximum shaft length shown on the Plans, whichever is greater.

COMMENTARY

The Engineer may modify the tip elevations shown on the Plans for the drilled shafts based on a) additional subsurface information obtained from exploratory borings performed during the construction contract, b) the results of the drilled shaft load test program, and/or c) the ground conditions identified during the drilled shaft excavation operations.

Blasting will only be permitted if specifically stated on the Plans or authorized in writing by the Engineer.

Sidewall overreaming shall be performed when the time for shaft excavation exceeds __ hours (measured from the beginning of excavation below the casing when casing is used) before the start of concrete placement. Sidewall overreaming shall also be performed when the sidewall of the hole is determined by the Engineer to have softened due to the excavation methods, swelled due to delays in the start of concrete placement, or degraded because of slurry cake buildup. Overreaming thickness shall be a minimum of 0.5 in. and a maximum of 3 in. Overreaming may be accomplished with a grooving tool, overreaming bucket, or other equipment approved by the Engineer. If overreaming is required as a result of the excavation time exceeding the time limit specified herein, or as a result of excavation methods not in compliance with the approved drilled shaft installation plan, the Contractor shall bear the costs associated with both sidewall overreaming and additional drilled shaft concrete related to overreaming.

COMMENTARY

A limit of 36 hours is often specified as the maximum time allowed for exposure of the drilled shaft excavation prior to the start of concrete placement. However, the actual time limit specified should be based on engineering judgment, considering the materials anticipated to be encountered and local practice. For materials which are resistant to degradation when exposed, such as cemented soils and durable rock, it may not be necessary to include a time limit in the Specifications.

Excavation to the foundation cap elevation shall be completed before drilled shaft construction begins, unless otherwise noted in the Contract Documents or approved by the Engineer. Any disturbance to the foundation cap area caused by shaft installation shall be repaired by the Contractor prior to placing the cap concrete.

When drilled shafts are to be installed in conjunction with embankment construction, the Contractor shall construct drilled shafts after placement of the embankment fill unless otherwise shown on the Contract Documents or approved by the Engineer. Drilled shafts installed prior to the completion of the embankment fill shall not be capped until the fill has been placed to the bottom of cap level.

COMMENTARY

Generally, embankment fill should be placed prior to installation of the drilled shafts to facilitate placement and compaction of the embankment fill, and to avoid potential damage to the drilled shafts that might occur if fill were to be placed and compacted near and between the drilled shafts. Installation of embankment fill and preloading of the underlying soils prior to installation of the drilled shafts may also reduce downdrag loads and lateral loads on the drilled shafts.

X.4.2 Drilled Shaft Excavation Protection

Drilled shaft excavations shall not be left open overnight unless cased full depth or otherwise protected against sidewall instability. The use of slurry to protect a *drilled* shaft during a drilling stoppage or overnight shutdown may be approved by the Engineer.

Casing of *drilled* shafts in stable rock formations during stoppages *will* not be required, *unless shown on the Plans or specified herein.*

COMMENTARY

An open excavation is defined as a *drilled* shaft that has not been filled with concrete, or temporarily backfilled with a material approved by the Engineer in accordance with Article X.2.2, or protected in accordance with Article X.4.3.

X.4.3 Drilled Shaft Excavation Protection Methods

The Contractor shall bear full responsibility for selection and execution of the method(s) of stabilizing and maintaining the *drilled* shaft excavation. The walls and bottom of the *drilled* shaft excavation shall be protected so that sidewall caving and bottom heave are prevented from occurring, and so that the soil adjacent to the *drilled* shaft is not disturbed. The Contractor may excavate the *drilled* shaft without excavation protection provided *the Contractor* can demonstrate that the soil/rock is stable within or above the *water table and* zones of seepage.

COMMENTARY

Project specification requirements may dictate that specific *drilled* shaft excavation protection methods should be used. For example, the contract may require that permanent casing be used if very soft soils are present that will not support the weight of the wet concrete when the casing is extracted, or if the foundations for an immediately adjacent structure are present and must be protected from movement.

Acceptable protection methods include the use of casing, drilling slurry, or both.

X.4.3.1 Temporary Casing Construction Method

In stable soils, the Contractor shall conduct casing installation and removal operations and *drilled* shaft excavation operations such that the adjacent soil outside the casing and *drilled* shaft excavation for the full height of the *drilled* shaft is not disturbed.

COMMENTARY

Disturbed soil is defined as soil whose geotechnical properties have been changed from those of the original in-situ soil, and whose altered condition adversely affects the performance of the *drilled* shaft foundation.

If the Contractor is utilizing casing that is adequately sealed into competent soils such that water cannot enter the excavation, the Contractor may, with the Engineer's approval, continue excavation in soils below the water table provided the water level within the casing does not rise or exhibit flow.

COMMENTARY

Temporary casing also includes surface (shallow) casing that is used to prevent sloughing and raveling of material into the drilled shaft excavation from the top of the hole. When extended a sufficient height above the ground surface, this surface casing can also serve as a part of the Contractor's safety measures to protect personnel from falling into the drilled shaft excavation.

As the temporary casing is withdrawn, a sufficient head of fluid concrete shall be maintained to ensure that water or slurry outside the temporary casing will not breach the column of freshly placed concrete.

Casing extraction shall be at a slow, uniform rate with the pull in line with the axis *of the* shaft. Excessive rotation of the casing shall be avoided to limit deformation of the reinforcing steel cage.

COMMENTARY

Movement of the casing by rotation, exerting downward pressure, and tapping to facilitate extraction, or extraction with a vibratory hammer is acceptable. The duration of vibration during casing extraction with a vibratory hammer should be limited in order to minimize potential segregation of the concrete.

The Contractor shall remove all temporary casings from the excavation as concrete placement is completed, unless permission has been received from the Engineer to leave specified temporary casings in place.

COMMENTARY

When temporary casing is placed in a predrilled hole there is a potential for loose material to be trapped in the annular void outside the casing. Loose material can also become trapped in the annular space between temporary and permanent casings. The presence of this loose material could cause defects at the perimeter of the completed shaft during extraction of the temporary casing as a result of material mixing with the concrete. In addition, the presence of this loose material may influence the performance of the drilled shaft during lateral loading. Attention must be given to these issues when developing the specific drilled shaft installation procedures. In addition, a drilled shaft integrity testing program should be considered to assess the structural integrity of the completed drilled shafts.

X.4.3.2 Permanent Casing Construction Method

Where permanent casing is specified, excavation shall conform to the specified outside diameter of the *drilled* shaft. After the casing has been filled with concrete, all void space occurring between the casing and *drilled* shaft excavation shall be filled with a material which approximates the geotechnical properties of the in-situ soils, in accordance with the *drilled* shaft installation plan specified in Article X.2.2 and as approved by the Engineer.

COMMENTARY

As outlined in *the commentary for* Article X.2.2, the backfill of accidental over-excavation outside the casing may require the use of materials which closely approximate the lateral response of the native soils.

In other cases the Engineer may require that foundation materials be sealed against evaporation or water introduction. In those cases, the Engineer may require that any annular space around a permanent casing be filled with structural grout.

Tops of permanent casings for the *drilled* shafts shall be removed to the top of the *drilled* shaft or finished ground line, whichever is lower, unless the top of permanent casing is shown in the Plans at a different elevation. For those *drilled* shafts constructed within a permanent body of water, tops of permanent casings for *drilled* shafts shall be removed to the low water elevation, unless *otherwise shown on the Plans or* directed otherwise by the Engineer.

Unless shown otherwise on the Plans, casing used for forming shafts installed through a body of water shall not be removed.

COMMENTARY

Drilled shafts constructed through a body of water will require the use of the casing as a form for the shaft concrete. In such cases, specifications often include provisions for removal of the top of the casing to or below low water level after the concrete reaches a specified minimum strength. Some Contracting Agency standard specifications also allow removal of the portion of the casing below water level. However, removal of the lower section of casing risks damage to the concrete and exposes any surface defects to potentially deleterious effects or accelerated corrosion in water. Often such defects are difficult to identify, or may not be suitably repaired or inspected under water. Based on these considerations, casing should not be removed below water level or below the bottom of the foundation cap, whichever is lower, unless this is required for compliance with an environmental or other regulatory requirement, and precautions are taken to maintain the structural integrity of the concrete.

X.4.3.3 Alternative Casing Methods

When approved by the Engineer, installation of casing using rotating or oscillating methods will be permitted. Use of this alternative casing method shall be in accordance with the equipment and procedures shown in the approved drilled shaft installation plan, and shall comply with all other requirements specified herein.

Drilled shaft casing shall be equipped with cutting teeth or a cutting shoe and installed by either rotating or oscillating the casing.

COMMENTARY

This alternative may be specified if vibratory placement or extraction of casing is not permitted.

Soils consisting of gravel and cobble mixtures, or matrix supported boulders where the matrix is loose and granular, tend to be susceptible to caving and sloughing, and usually require casing to stabilize the *drilled* shaft side walls. These materials also make vibratory casing installation very difficult and risky for both the Contracting Agency and the Contractor. In such cases, the installation of temporary and/or permanent casing by either a rotating or an oscillating method may be required.

X.4.3.4 Slurry

The Contractor shall use slurry in accordance with Articles X.3.4 through X.3.6 to maintain a stable excavation during excavation and concrete placement operations once water begins to enter the *drilled* shaft excavation and remain present

The Contractor shall use slurry to maintain stability during *drilled* shaft excavation and concrete placement operations in the event water begins to enter the *drilled* shaft excavation at a rate of greater than 12.0 in./hour; or if the Contactor is not able to restrict the amount of water in the *drilled* shaft to less than 3.0 in. prior to *concrete placement*, or to equilibrate water pressure on the sides and base of the *drilled* shaft excavation when groundwater is encountered or anticipated based on the available subsurface data.

COMMENTARY

Many situations will require the Contractor to utilize both slurry and casing techniques in the same hole.

X.4.3.4.1 Slurry Technical Assistance

If slurry is used, the manufacturer's representative, as identified to the Engineer in accordance with Article X.2.3, shall:

- provide technical assistance for the use of the slurry,
- be at the site prior to introduction of the slurry into a drilled hole, and
- remain at the site during the construction and completion of a minimum of one *drilled* shaft to adjust the slurry mix to the specific site conditions.

After the manufacturer's representative is no longer present at the site, the Contractor's employee trained in the use of the slurry, as identified to the Engineer in accordance with Article X.2.3, shall be present at the site throughout the remainder of *drilled* shaft slurry operations for this project to perform the duties specified above.

X.4.3.4.2 Minimum Level of Slurry in the Excavation

Where slurry is used to maintain a stable excavation, the slurry level in the excavation shall be maintained to obtain hydrostatic equilibrium throughout the construction operation at a height required to provide and maintain a stable hole, but not less than 5.0 ft above the water table *or surface of surrounding water body if at an offshore location*.

The Contractor shall provide casing, or other means, as necessary to meet these requirements.

The slurry level shall be maintained above all unstable zones a sufficient distance to prevent bottom heave, caving or sloughing of those zones.

COMMENTARY

Recommended slurry levels are as follows:

- not less than 5.0 ft for mineral slurries,
- not less than 10.0 ft for water slurries, and
- not less than 10.0 ft for polymer slurries, except when a lesser dimension is specifically recommended by the slurry manufacturer for the site conditions and construction *methods*.

Artesian conditions may require slurry levels even greater than noted for the above slurry types.

Throughout all stops in *drilled* shaft excavation operations, the Contractor shall monitor and maintain the slurry level in the excavation the greater of the following elevations:

- no lower than the water level elevation outside the *drilled* shaft, or
- an elevation as required to provide and maintain a stable hole.

COMMENTARY

When drilling fluid is used to maintain the stability of the hole, such as in clean, granular soils, a positive hydrostatic pressure should be maintained within the drilled shaft excavation during any interruptions or stops in the drilled shaft excavation operation to reduce the risk of instability of the drilled shaft excavation. Staff and pumping equipment should be provided to maintain the fluid levels above the groundwater level, or above the adjacent water surface at offshore locations, during such periods.

X.4.3.4.3 Cleaning Slurry

The Contractor shall clean, re-circulate, de-sand, or replace the slurry, as needed, in order to maintain the required slurry properties in accordance with Articles 5.3.4 and 5.3.5. Sand content shall be within specified limits as specified in the Contract, prior to concrete placement

X.4.4 Obstructions

When obstructions are encountered, the Contractor shall notify the Engineer promptly. When efforts to advance past the obstruction to the design *drilled* shaft tip elevation result in a reduction in the rate of advance and/or change in approved means and methods relative to the approved *drilled* shaft installation plan, then the Contractor shall remove, bypass or break up the obstruction under the provisions of Article X.5.1.3. *Blasting will not be permitted unless approved in writing by the Engineer.*

Drilling tools that are lost in the excavation will not be considered obstructions, and shall be promptly removed by the Contractor. All costs due to lost tool removal will be borne by the Contractor including, but not limited to, costs associated with the repair of hole degradation due to removal operations or an excessive time that the hole remains open.

COMMENTARY

An obstruction is defined as a specific object (including, but not limited to, boulders, logs, and man-made objects) encountered during the *drilled* shaft excavation operation which prevents or hinders the advance of the *drilled* shaft excavation.

A focused effort should be made during the project design phase to determine if obstructions may be encountered during drilled shaft excavation. Special notes should be included in the Plans to alert the Contractor of the type, approximate size range, and likely locations of obstructions. The presence of obstructions should also be documented in the geotechnical or foundation report for the project.

If the *Contracting* Agency chooses to limit obstruction removal to "unknown obstructions" it places a heavy burden on the Foundation Report to accurately describe the obstructions which a contractor should anticipate.

X.4.5 Protection of Existing Structures

The Contractor shall control operations to prevent damage to existing structures, utilities, *roadways and other facilities.* Preventative measures shall include, but are not limited to, selecting construction methods and procedures that will prevent excessive caving of the *drilled* shaft excavation, and monitoring and controlling the vibrations from the driving of casing or sheeting, drilling of the shaft, or from blasting, if permitted.

COMMENTARY

This Section will be used for site specific issues such as shallow foundations adjacent to drilled shaft work or adjacent vibration sensitive installations. The *Contracting* Agency may choose to specify casing installation in advance of excavation or may restrict the amount of vibration a contractor may use to install or remove casing or perform drilling operations.

The specific monitoring requirements for structures impacted by drilled shaft construction should be assessed individually for each project. If monitoring is determined to be necessary, a preconstruction survey of existing facilities should be performed to establish baseline data, including ambient vibration levels and existing structure defects. When vibrations are to be monitored, it is preferable for the Contracting Agency or Engineer to engage the services of a professional vibrations consultant to monitor, record and report vibration levels during drilled shaft construction. Alternatively, the Contract Documents can include provisions requiring the Contractor to engage a professional vibrations consultant for this monitoring program.

X.4.6 Slurry Sampling and Testing

Mineral slurry and polymer slurry shall be mixed and thoroughly hydrated in slurry tanks, lined ponds, or storage areas. The Contractor shall draw sample sets from the slurry storage facility and test the samples for conformance with the appropriate specified material properties before beginning slurry placement in the drilled hole. Slurry shall conform to the quality control plan included in the *drilled* shaft installation plan in accordance with Article X.2.2 and approved by the Engineer. A sample set shall be composed of samples taken at mid-height and within 2.0 ft of the bottom of the storage area.

COMMENTARY

Lined ponds should generally not be permitted for mixing or storing slurry.

The Contractor shall sample and test all slurry in the presence of the Engineer, unless otherwise *approved by the Engineer*. The date, time, names of the persons sampling and testing the slurry, and the results of the tests shall be recorded. A copy of the recorded slurry test results shall be submitted to the Engineer at the completion of each *drilled* shaft, and during construction of each *drilled* shaft when requested by the Engineer.

Sample sets of all slurry, composed of samples taken at mid-height and within 2.0 ft of the bottom of the *drilled* shaft, shall be taken and tested during drilling as necessary to verify the control of the properties of the slurry. As a minimum, sample sets of polymer slurry shall be taken and tested at least once every 4.0 *hours* after beginning its use during each shift.

Sample sets of all slurry, as specified, shall be taken and tested immediately prior to placing concrete.

The Contractor shall demonstrate to the satisfaction of the Engineer that stable conditions are being maintained. If the Engineer determines that stable conditions are not being maintained, the Contractor shall immediately take action to stabilize the shaft. The Contractor shall submit a revised *drilled* shaft installation plan that addresses the problem and prevents future instability. The Contractor shall not continue with *drilled* shaft construction until the damage which has already occurred is repaired in accordance with the Specifications, and until receiving the Engineer's approval of the revised *drilled* shaft installation plan.

X.4.7 *Drilled* Shaft Excavation Inspection

The Contractor shall use appropriate means, such as a cleanout bucket, air lift, or hydraulic pump, to clean the bottom of the excavation of all *drilled* shafts. For wet drilled shaft excavations in soils, the base of the *drilled* shaft excavation shall be covered with not more than 3.0 in. of sediment or loose or disturbed material just prior to placing concrete. For dry drilled shaft excavations in soils, the base of the excavation shall be covered with not more than 1.5 in. sediment or loose or disturbed material just prior to placing concrete. For wet or dry drilled shaft excavations in rock, the base of the excavation shall be covered with not more than 0.5 in. for 50 percent of the base area of sediment or loose or disturbed material just prior to placing concrete.

The excavated *drilled* shaft *will* be inspected and approved by the Engineer prior to proceeding with construction. The bottom of the excavated *drilled* shaft shall be sounded with an airlift pipe, a tape with a heavy weight attached to the end of the tape, *a borehole camera with visual sediment depth measurement gauge,* or other means acceptable to the Engineer to determine that the *drilled* shaft bottom meets the requirements in the Contract.

COMMENTARY

Alternative bottom cleanliness criteria specified by several Contracting Agencies limit the average thickness of sediments on the shaft base to 0.5 in., and the maximum thickness to 1.5 in. These alternative criteria are considered more appropriate for shafts that rely on end bearing for a large portion of the required nominal resistance of the drilled shaft.

The amount of sediment left on the base of the *drilled* shaft can be determined by using a weighted tape and bouncing it on the bottom of the *drilled* shaft. If the weight strikes the bottom of the excavation with an immediate stop, the *drilled* shaft has little or no sediment. If the weight slows down and sinks to a stop, then excessive sediment exists.

The depth of sediment determined with the use of a weighted tape is subject to interpretation, and use of a weighted tape is generally appropriate when a firm bottom can be achieved by the selected excavation and bottom cleaning methods. If further confirmation or documentation of bottom cleanliness is desired, or if sediments are thicker than can be evaluated with the use of a weighted tape, consideration should be given to use of camera techniques for inspection of the drilled shaft bottom. Camera inspection methods should include means for visually measuring the thickness of loose sediments with a sediment depth gauge. Camera inspection equipment is currently available for use in slurry filled holes. If cleanliness of less than 2.0 in. of loose material is required, the inspection must utilize camera techniques that enable visual inspection.

The cleanliness of the *drilled* shaft base is a requirement not only for end bearing and settlement considerations but also to obtain an uncontaminated concrete pour.

Mucker buckets, airlifts, and special *buckets* that have gates that can be opened and closed are typically used *for bottom cleaning*. Flocculents *to precipitate soil particles from slurry* may be required for wet excavations.

Visual inspection by personnel in the hole should generally be avoided based on safety considerations. If entry is required, measures must be implemented to provide for the safety of the inspector.

X.4.8. Assembly and Placement of Reinforcing Steel

Prior to and during fabrication of the steel reinforcing cage, the reinforcing bars shall be supported off the ground surface, and shall be protected from contamination with mud and other deleterious materials.

The Contractor shall show bracing and any extra reinforcing steel required for fabrication of the cage on the shop drawings. The Contractor will be responsible for engineering the temporary support and bracing of reinforcing cages to ensure that they maintain their planned configuration during assembly, transportation, and installation. As a minimum:

- At least 4 vertical bars of each cage, equally spaced around the circumference, shall be tied at all reinforcement intersections with double wire ties.
- At least 25 percent of remaining reinforcement intersections in each cage shall be tied with single wire ties. Tied intersections shall be staggered from adjacent ties.
- Bracing shall be provided to prevent collapse of the cage during assembly, transportation, and installation.

COMMENTARY

Allowable tolerance of the reinforcing cage is based on minimum CRSI intersection tie requirements, plus whatever additional ties and bracing necessary to maintain the cage shape.

Recommended concrete cover to reinforcing steel:

Drilled Shaft Diameter	**Minimum Concrete Cover**
Less than or equal to 3.0 ft	3.0 in.
Greater than 3.0 ft and less than 5.0 f	4.0 in.
5.0 ft or larger	6.0 in.

Successful completion of these minimum baseline requirements for reinforcement cage will *not* relieve the Contractor of full responsibility for engineering the temporary support and bracing of the cages during construction.

The reinforcement shall be carefully positioned and securely fastened to provide the minimum clearances *specified herein or shown on the Plans*, and to ensure that no displacement of the reinforcing steel cage occurs during placement of the concrete.

Splicing of the reinforcement cage during placement of the cage in the shaft excavation will not be permitted unless otherwise shown on the Plans or approved by the Engineer.

If the reinforcing cage is spliced during placement of the cage into the drilled shaft excavation, the splice details and location of the splices shall be in accordance with the Plans and the approved drilled shaft installation plan. In addition, the work shall be performed within the time limits specified in Article X.4.1.

The steel reinforcing cage shall be securely held in position throughout the concrete placement operation. The reinforcing steel in the *drilled* shaft shall be tied and supported so that the location of the reinforcing steel will remain within allowable tolerance. Concrete spacers or other approved non-corrosive spacing devices shall be used at sufficient intervals (near the bottom, the top and at intervals not exceeding 10.0 ft vertically) to ensure concentric spacing for the entire cage length. The number of spacers required at each level shall be one spacer for each 1.0 ft of excavation diameter, with a minimum of four spacers at each level. The spacers shall be of adequate dimension to ensure an annular space between the outside of the reinforcing cage and the side of the excavation along the entire length of the *drilled* shaft as shown in the Plans. Acceptable feet made of plastic or concrete (bottom supports) shall be provided to ensure that the bottom of the cage is maintained at the proper distance above the base of the excavation, unless the cage is suspended from a fixed base during the concrete pour.

Bracing steel that constricts the interior of the reinforcing cage shall be removed after lifting the cage if freefall concrete or wet tremie methods of concrete placement are to be used.

The elevation of the top of the steel cage shall be checked before and after the concrete is placed. If the upward displacement of the rebar cage exceeds 2.0 in., or if the downward displacement exceeds 6.0 in., the drilled shaft will be considered defective. Corrections shall be made by the Contractor to the satisfaction of the Engineer. No additional drilled shafts shall be constructed until the Contractor has modified the rebar cage support in a manner satisfactory to the Engineer.

X.4.9 Concrete Placement, Curing and Protection

Concrete placement shall commence as soon as possible after completion of drilled shaft excavation by the Contractor and inspection by the Engineer. Immediately prior to commencing concrete placement, the *drilled* shaft excavation and the properties of the slurry (if used) shall conform to Articles X.3.4 through X.3.6. Concrete placement shall continue in one operation to the top of the *drilled* shaft, or as shown in the Plans.

If water is not present (a dry shaft), the concrete shall be deposited through the center of the reinforcement cage by a method which prevents segregation of aggregates on the reinforcement cage. The concrete shall be placed such that the free-fall is vertical down the center of the *drilled* shaft without hitting the sides, the steel reinforcing bars, or the steel reinforcing bar cage bracing.

COMMENTARY

Research has demonstrated that virtually unlimited free-fall is acceptable if the concrete mix is cohesive and contains relatively small maximum-sized coarse aggregate. However, the height of free fall should be controlled to reduce the risk of the concrete impacting the reinforcing steel cage. The specified maximum height of free fall should consider the diameter of the shaft excavation and the diameter of the reinforcing cage. For very large-diameter shafts with large-diameter cages, where the danger of striking the cage is reduced, it may be permissible to permit free-fall to a height of as much as 80 ft, consistent with the maximum free fall evaluated in tests by Baker (1960). Free fall concrete can be guided to the center of the shaft with the use of a centering hopper.

If water exists in amounts greater than 3.0 in. in depth or enters at a rate of more than 12.0 in. per hour, then the *drilled* shaft excavation shall be filled with slurry to at least the level specified in Article X.4.3.4.2 and concrete placed by tremie methods.

COMMENTARY

A practical definition of a dry shaft is when the amount of standing water in the base of the shaft prior to concreting is less than or equal to 3.0 in., and water is entering the shaft at a rate of less than 12.0 in. per hour.

The elapsed time for concrete placement shall not exceed the time limit defined in the approved drilled shaft installation plan and demonstrated by a successful technique shaft or test shaft. For wet placement methods, the concrete placement time shall commence at the mixing of the concrete and extend through to the completion of placement of the concrete in the drilled shaft excavation, including removal of any temporary casing. Prior to concrete placement, the Contractor shall provide test results of both a trial mix and a slump loss test conducted by an approved testing laboratory using approved methods to demonstrate that the concrete meets this defined placement time limit. The concrete mix shall maintain a slump of not less than the minimum value specified in Article X.3.1 over the defined placement time limit as demonstrated by trial mix and slump loss tests. The trial mix and slump loss tests shall be conducted at ambient temperatures appropriate for site conditions. Ambient air temperature at the time of concrete placement shall not be greater than the ambient temperature at the time of the concrete trial tests and slump loss tests.

COMMENTARY

For smaller shafts commonly used in practice, many State DOT Standard Specification specify a time limit for concrete placement (e.g. 2 hours placement into the shaft plus transit time), with provisions allowing additional time if successfully demonstrated by testing of a trial mix. Deep, large diameter drilled shafts typically require a concrete placement time that greatly exceeds these commonly specified time limits.

The concrete must maintain the specified minimum slump for the full placement period, which includes transportation of the concrete to the shaft location, placement of the concrete into the shaft, and removal of any temporary casing. This minimum slump is required to assure that the concrete remains sufficiently plastic to flow through the reinforcement cage and to fill areas vacated by the removal of temporary casing. The specified minimum slump value is the lowest slump at which adequate fluid pressures can be assumed to develop against the sides of the hole.

Admixtures such as water reducers, plasticizers, and retarders shall not be used in the concrete mix unless permitted in the Contract Documents and detailed in the approved drilled shaft installation plan. All admixtures, when approved for use, shall be adjusted for the conditions encountered on the job so the concrete remains in a workable plastic state throughout the defined placement time limit.

COMMENTARY

For longer concrete placement times, retarders and additives will likely be required to maintain the specified minimum concrete slump for the longer time period.

Throughout the underwater concrete placement operation, the discharge end of the tube shall remain submerged in the concrete at least 5.0 ft and the tube shall always contain enough concrete to prevent water from entering. The concrete placement shall be continuous until the work is completed, resulting in a seamless, uniform shaft. If the concrete placement operation is interrupted, the Engineer may require the Contractor to prove by core drilling or other tests that the *drilled* shaft contains no voids or horizontal joints. If testing reveals voids or joints, the Contractor shall repair them or replace the *drilled* shaft at no expense to the Contracting Agency. Responsibility for coring *and* testing costs, and calculation of time extension, shall be in accordance with Article X.4.12.

COMMENTARY

A greater penetration of the tremie pipe into the concrete is desirable to reduce the risk of inadvertently pulling the tremie out of the concrete. Consideration should be given to increasing the minimum tremie penetration to 10.0 ft into the concrete, particularly for fluid mixes that will flow readily with the increased penetration.

Technique shafts provide an opportunity to assess concrete placement time, as well as concrete placement procedures.

In cases where it is possible to pour tremie placed *drilled* shafts to *the* ground surface, the Contractor should consider placing concrete until a minimum of 18.0 in. of concrete, measured vertically, has been expelled to eliminate contaminates in the top of the shaft pour.

Because of the nature of drilled shaft mix designs, it is unnecessary to vibrate the concrete. *In addition, vibrating the concrete is undesirable since it may cause segregation of the aggregate.*

Before placing any fresh concrete against concrete deposited in water or slurry (construction joint), the Contractor shall remove all scum, laitance, loose gravel, and sediment on the surface of the concrete deposited in water or slurry, and chip off any high spots on the surface of the existing concrete that would prevent any steel reinforcing bar cage from being placed in the position required by the Plans.

The Contractor shall complete a concrete yield plot for each wet shaft poured by tremie methods. This yield plot will be submitted to the *Contracting* Agency within 24.0 hours of completion of the concrete pour

The Contractor shall not perform *casing installation or drilled* shaft excavation operations within *a clear distance of* three diameters of a newly poured *drilled* shaft within 24.0 hours of the placement of concrete and only when the concrete has reached a minimum compressive strength of 1,800 psi.

X.4.10 Tremies

When placing concrete underwater, the Contractor shall use a concrete pump or gravity tremie. A tremie shall have a hopper at the top that empties into a watertight tube at least 8.0 in. in diameter. If a pump is used, a watertight tube shall be used with a minimum diameter of 4.0 in., except as noted herein. The discharge end of the tube on the tremie or concrete pump *line* shall include a device to seal out water while the tube is first filled with concrete. In lieu of a seal at the discharge end of the pipe, the Contractor may opt to place a “pig” or “rabbit” in the hopper prior to concrete placement that moves through the tremie when pushed by the concrete, forcing water or slurry from the tremie pipe.

The hopper and tubes shall not contain aluminum parts that will have contact with the concrete. The inside and outside surfaces of the tubes shall be clean and smooth to allow both flow of concrete and the unimpeded withdrawal of the tube during concrete placement.

COMMENTARY

An alternative specification that should be considered requires a minimum tremie pipe inside diameter of 10.0 in. and a minimum pump line inside diameter of 5.0 in. These larger diameters facilitate flow of concrete and reduce the risk of plugging of the pipe during concrete placement.

A pig or rabbit is a flexible device that fills the entire cross section (at least 110 percent) of the tremie tube and creates an impermeable separation between the concrete in the tremie and the slurry.

A tremie (with pig or end cap seal) or pump extension should be used for all wet placements so that the water does not mix with the concrete as it is being placed in the excavation. Trapped air in the pump line or tremie will cause mixing of the concrete and any available water *or slurry.*

Mark the tremie pipe *(or pump line)* so that tremie insertion and concrete head may be determined. In addition, it is good practice to know the volume placed per stroke of the concrete pump to validate the concrete head.

Reinsertion of a tremie or pump *line* implies a loss of head. Removal of contaminated concrete is advisable, and coring or Cross-hole Sonic Logging (CSL) testing should be done.

X.4.11 Drilled Shaft Construction Tolerances

Drilled shafts shall be constructed so that the center of the poured shaft at the top of the *drilled* shaft or mudline, whichever is lower, is within the following horizontal tolerances:

***Drilled* Shaft Diameter**	**Tolerance**
Less than or equal to 2.0 ft	3.0 in.
Greater than 2.0 ft and less than 5.0 ft	4.0 in.
5.0 ft or larger	6.0 in.

COMMENTARY

The top of shaft horizontal tolerance presented in this specification represents a practical bound that is considered achievable for all drilled shaft projects. Lower values can be considered if warranted by design requirements, but this may require special techniques and may result in higher costs.

Lateral plan deviation less than specified should be shown on the Contract Plans.

In cases where it is not practical to determine the location of the top of the drilled shaft or shaft location at the mudline, such as at shafts for underwater footings, the top of the casing can be used for setting and checking the location of the shaft. For these situations, the foundation design should consider a greater offset tolerance at the top of the drilled shaft than the values specified.

Drilled shafts in soil shall be within 1.5 percent of plumb. Drilled shafts in rock shall be within 2.0 percent of plumb. Plumbness shall be measured from the top of poured *drilled* shaft elevation or mudline, whichever is lower.

During drilling or excavation of the *drilled* shaft, the Contractor shall make frequent checks on the plumbness, alignment, and dimensions of the *drilled* shaft. Any deviation exceeding the allowable tolerances shall be corrected with a procedure approved by the Engineer.

Drilled shaft steel reinforcing bars shall be no higher than 6.0 in. above or 3.0 in. below the plan elevation.

The reinforcing cage shall be concentric with the drilled shaft excavation within a tolerance of 1.5 in.

The top elevation of the completed drilled shaft shall have a tolerance of plus one inch or minus 3.0 in.

The diameter of the drilled shaft shall not be less than the diameter shown on the Plans.

Tolerances for casings shall be in accordance with American Pipe Institute tolerances applicable to regular steel pipe.

Drilled shaft excavations and completed drilled shafts not constructed within the required tolerances will be considered defective. The Contractor shall be responsible for correcting all defective drilled shafts to the satisfaction of the Engineer. Materials and work necessary, including engineering analysis and redesign, to complete corrections for out-of-tolerance drilled shafts shall be furnished without either cost to the Contracting Agency or an extension of the completion date of the project. Redesign drawings and computations submitted by the Contractor shall be signed by a registered Professional Engineer licensed in the State of _____.

COMMENTARY

When a completed drilled shaft excavation exceeds the specified tolerances, the Contractor should be required to propose, develop, and, after approval, implement corrective treatment. The Engineer should not direct the work. Typical corrective treatments include:

(a) Overdrill the drilled shaft excavation to a larger diameter to permit accurate placement of the reinforcing steel cage with the required minimum concrete cover.

(b) Increase the number and/or size of the steel reinforcement bars.

(c) Drill out the green concrete and reform the hole.

(d) Downgrade the resistance value for the drilled shaft.

(e) Install one or more additional drilled shafts to replace or supplement the out-of-tolerance shaft.

In addition, the Engineer can perform analyses to determine if the out-of-tolerance shaft can be accepted without corrective measures.

X.4.12 Integrity Testing

Cross-hole Sonic Logging (CSL) testing shall be performed on *drilled* shafts as specified in the Contract. The Contractor shall accommodate the CSL testing by furnishing and installing access tubes in accordance with Article X.3.7.

COMMENTARY

CSL testing is used as a regular inspection method for wet placement shafts using tremie concrete methods. Other Non-Destructive Testing (NDT) methods available include Gamma-Gamma (GG) testing, Pulse Echo Testing, *and Thermal Integrity Profiling (TIP). Specifications have been included herein for CSL testing since CSL testing is a common NDT method for drilled shafts. See Chapter 16 of this Manual for a discussion of the details and procedures for other, less common methods for integrity testing.*

The Contract Documents should note the extent of CSL testing for the Project and the organization (Contractor, Engineer, or Contracting Agency) responsible for performing CSL testing.

The Contractor shall install access tubes for CSL testing in all drilled shafts, except as otherwise noted *herein*, to permit access for the CSL test probes. If, in the opinion of the Engineer, the condition of the *drilled* shaft excavation permits *drilled* shaft construction in the dry, the Engineer may specify that the testing be omitted.

COMMENTARY

CSL testing should only be used for *drilled* shafts placed in the dry where visual inspection indicates that irregularities in concrete placement may have occurred, *and at all technique shafts and load test shafts.*

The Contractor shall securely attach the access tubes to the interior of the reinforcement cage of the *drilled* shaft. One access tube shall be furnished and installed for each foot of *drilled* shaft diameter, rounded to the nearest whole number, *unless otherwise* shown in the Plans. A minimum of three tubes *will* be required *for each drilled shaft*. The access tubes shall be placed around the *drilled* shaft, inside the spiral or hoop reinforcement and 3.0 in. clear of the vertical reinforcement, at a uniform spacing measured along the circle passing through the centers of the access tubes. If these minimums cannot be met due to close spacing of the vertical reinforcement, then the access tubes shall be bundled with the vertical reinforcement.

COMMENTARY

The tubes can be placed either *adjacent to vertical* reinforcing steel or midspan between vertical reinforcement. There are concerns that CSL tubes placed midspan will effectively negate the clear spacing requirements specified in Article X.3.2 for the reinforcing steel. There are competing concerns that CSL tubes bundled with large reinforcing steel bundles will create a large blockage for proper flow of concrete. These issues must be resolved on a case-by-case basis.

If the CSL tubes are bundled with the vertical reinforcement, care needs to be taken to assure the concrete remains fluid throughout the concrete placement period.

If trimming the cage is required and access tubes for CSL 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.

COMMENTARY

If the reinforcing steel does not extend to the bottom of the drilled shaft, the CSL tubes should be extended to the *drilled* shaft bottom. *In such cases, a sufficient number of reinforcing bars should be extended to the bottom of the drilled shaft excavation to secure the CSL tubes in position.*

The access tubes shall be installed in straight alignment and as near to parallel to the vertical axis of the reinforcement cage as possible. The access tubes shall extend from the bottom of the *drilled* shaft to at least 2.0 ft above the top of the *drilled* shaft. Splice joints in the access tubes, if required to achieve full length access tubes, shall be watertight. The Contractor shall clear the access tubes of all debris and extraneous materials before installing the access tubes. Care shall be taken to prevent damaging the access tubes during reinforcement cage installation and concrete placement operations in the *drilled* shaft excavation.

The access tubes shall be filled with potable water before concrete placement, and the top watertight threaded caps shall be reinstalled.

Prior to performing any CSL testing operations specified herein, the Contractor shall remove the concrete at the top of the *drilled* shaft down to sound concrete.

When the Contract indicates that the Contracting Agency will be responsible for CSL testing, the Contracting Agency will perform CSL testing and analysis on all completed *drilled* shafts designated for testing by the Engineer. The Contractor shall give at least 48 hours notice to the Engineer of the time the concrete in each *drilled* shaft will be sufficiently cured to allow for cross-hole sonic log testing.

When the Contract requires CSL testing to be performed by a testing firm engaged by the Contractor, CSL testing and reporting procedures shall be in accordance with ASTM D4428/4428M, unless otherwise specified herein. The Contractor's testing firm shall perform CSL testing on all drilled shafts designated by the Engineer or required by Contract.

COMMENTARY

The *Contracting* Agency may opt to require the Contractor to provide the Integrity Testing and provide the results to the *Contracting* Agency for its approval. *For this option, the Specifications should require the Contractor to engage a qualified testing consultant to perform the integrity testing, and include provisions defining the minimum qualifications of the integrity testing consultant and the individual(s) performing the tests. The Specifications also need to include requirements for performing the tests (including the tube pairs to be tested, and the depth frequency of testing), and provisions for test reports including information and data plots to be presented in the report.*

The testing shall be performed after the *drilled* shaft concrete has cured at least 96 hours. Additional curing time prior to testing may be required if the *drilled* shaft concrete contains admixtures, such as set retarding admixture or water reducing admixture. The additional curing time prior to testing required under these circumstances shall not be grounds for additional compensation or extension of time to the Contractor. No subsequent construction shall be performed on the completed *drilled* shaft until the CSL tests are approved and the *drilled* shaft accepted *by the Engineer*. The CSL shall be completed within seven days of *concrete* placement of the shaft.

COMMENTARY

To reduce the risk of debonding the CSL tubes, consideration should be given to specify a maximum time for initial CSL testing after concrete placement. The maximum time should consider the type of material used for the CSL tubes.

After placing the *drilled* shaft concrete and before beginning the CSL testing of a *drilled* shaft, the Contractor shall inspect the access tubes. Each access tube that the test probe cannot pass through shall be replaced, at the Contractor's expense, with a 2.0 in. diameter hole cored through the concrete for the entire length of the *drilled* shaft. Unless directed otherwise by the Engineer, cored holes shall be located approximately 6.0 in. inside the reinforcement and shall not damage the *drilled* shaft reinforcement. Descriptions of inclusions and voids in cored holes shall be logged and a copy of the log shall be submitted to the Engineer. Findings from cored holes shall be preserved, identified as to location, and made available for inspection by the Engineer.

COMMENTARY

If a single tube is blocked, the Engineer may perform CSL testing on the remaining tubes. If no anomalies are noted, the Engineer may waive the requirement to provide the cored alternative hole.

The Engineer *will* determine final acceptance of each *drilled* shaft, based on the CSL test results and analysis for the tested shafts and a review of the visual inspection reports for the subject *drilled* shaft, and will provide a response to the Contractor within three working days after receiving the test results and analysis submittal.

COMMENTARY

In cases where a defect is suspected, but the potential impact of the defect is uncertain, the Engineer's response can note the need for additional time to perform engineering analyses for the shaft. Such analyses would consider the location and likely extent of the suspected defect. With such analyses it is possible that the drilled shaft may be determined to meet performance requirements and be acceptable even if a zone of poor quality concrete is present.

The Engineer may approve the continuation of *drilled* shaft construction prior to approval and acceptance of the first shaft if the Engineer's observations of the construction of the first shaft are satisfactory, including, but not limited to, conformance to the *drilled* shaft installation plan, as approved by the Engineer; and the Engineer's review of Contractor's daily reports and inspector's daily logs concerning excavation, steel reinforcing bar placement, and concrete placement are satisfactory.

If the Engineer determines that the concrete placed under slurry for a given *drilled* shaft is structurally inadequate, that *drilled* shaft *will* be rejected. The placement of concrete under slurry shall be suspended until the Contractor submits to the Engineer written changes to the methods of *drilled* shaft construction needed to prevent future structurally inadequate *drilled* shafts, and receives the Engineer's written approval of the submittal.

If *the Engineer determines that additional investigation is necessary, or if* the Contractor requests, the Engineer may direct that additional testing be performed at a *drilled* shaft. At the Engineer's request, the Contractor shall drill a corehole in any questionable quality *drilled* shaft (as determined from CSL testing and analysis or by observation of the Engineer) to explore the *drilled* shaft condition. *The number, locations, diameter and depth of the core holes and lengths of individual core runs will be determined by the Engineer. Coring procedures shall avoid damage to the steel reinforcement. Descriptions of inclusions and voids in cored holes shall be logged and a copy of the log shall be submitted to the Engineer. Recovered core shall be preserved in suitably labeled wood core boxes, identified as to location and depth, and made available for inspection by the Engineer. The Engineer may direct water pressure testing in the core holes, and/or unconfined compression testing and other laboratory testing on selected samples from the concrete core.*

Prior to beginning coring, the Contractor shall submit the method and equipment to be used to drill and remove cores from *drilled* shaft concrete to the Engineer, and shall not begin coring until it has received the Engineer's written approval. The coring method and equipment shall provide for complete core recovery and shall minimize abrasion and erosion of the core.

COMMENTARY

The recovery system most often used to ensure undamaged core recovery is the triple barrel system.

If subsequent testing at a *drilled* shaft indicates the presence of a defect(s) in the *drilled* shaft, the testing costs and the delay costs resulting from the additional testing shall be borne by the Contractor. If this additional testing indicates that the *drilled* shaft has no defect, the testing costs and the delay costs resulting from the additional testing *will* be paid by the Contracting Agency, and, if the *drilled* shaft construction is on the critical path of the Contractor's schedule, a time extension equal to the delay created by the additional testing *will* be granted.

COMMENTARY

A defect is defined as a feature which will result in inadequate or unsafe performance under strength, service, and extreme event loads, or inadequate serviceability over the design life of the drilled shaft, in the opinion of the Engineer.

It is suggested that the above definition of "defect" be replaced with the following: "A defect includes any void, discontinuity, deficient concrete strength, inclusion or crack within the drilled shaft concrete that, in the opinion of the Engineer, requires further investigation, whether or not it is subsequently determined to represent an inadequate or unsafe condition for the completed drilled shaft." This suggested revision is consistent with the approach discussed in Chapter 17 of this Reference Manual. This revision gives the responsibility for the cost of further investigation to the Contractor whenever shaft installation results in a defect in the shaft concrete.

For all *drilled* shafts determined to be unacceptable, the Contractor shall submit a plan for further investigation and remedial action to the Engineer for approval. All modifications to the dimensions of the *drilled* shafts, as shown in the Plans, required by the investigation and the remedial action plan shall be supported by calculations and working drawings. *All investigation and remedial correction procedures and designs shall be prepared by a registered Professional Engineer licensed in the State of _____, and submitted to the Engineer for approval.* The Contractor shall not begin repair operations until receiving the Engineer's *written* approval of the investigation and remedial action plan.

All access tubes and cored holes shall be dewatered and filled with grout after tests are completed and the *drilled* shaft is accepted *by the Engineer*. The access tubes and cored holes shall be filled using grout tubes that extend to the bottom of the tube or hole or into the grout already placed.

COMMENTARY

If a defect is discovered, it is recommended that three dimensional tomography be conducted to better define the extent of the defect.

When remediation of a drilled shaft is determined to be necessary, the CSL tubes should be maintained for post-remediation testing, and not grouted until completion of the necessary remediation measures and final written acceptance of the drilled shaft by the Engineer.

X.5 DRILLED SHAFT LOAD TESTS

Test shafts shall be installed at the locations shown on the Plans unless otherwise directed or approved by the Engineer.

COMMENTARY

See Chapter 13 of this Manual for descriptions and discussion of drilled shaft load test methods.

Test shafts shall be installed to the same dimensions, details and elevations shown on the Plans, and shall be installed using the same equipment and installation procedures proposed for installation of the foundation drilled shafts.

If the equipment or procedures are changed following the completion of load testing, the Contractor shall install additional load test shafts, and conduct additional load tests as directed by the Engineer at no additional cost to the Contracting Agency.

All load testing shall be completed and the results evaluated by the Engineer before installing any production drilled shafts, unless otherwise authorized by the Engineer.

X.5.1 Static Load Tests

Static load tests shall be performed in accordance with the procedures specified in ASTM D1143/D1143M for static compression load tests, and in accordance with ASTM D3689/D3689M for static tension load tests.

COMMENTARY

ASTM D1143/D1143M and D3689/D3689M describe the complete procedures for performing axial compression load tests and axial tensile load tests, respectively.

X.5.2 Force Pulse (Rapid) Load Tests

Force pulse (rapid) load tests shall be performed in accordance with the procedures specified in ASTM D7383.

COMMENTARY

ASTM D7383 specifies two alternative procedures, including "Procedure A" where the compression force pulse is applied at the top of the drilled shaft using a combustion gas pressure, and "Procedure B" where the compression force pulse is applied by dropping a mass onto the cushioned top of the drilled shaft. Each of these methods estimates resistance along the drilled shaft by evaluating the dynamic response of the drilled shaft and the dynamic characteristics of the supporting ground.

X.5.3 Bi-Directional Load Cell Testing

Bi-directional load cell tests shall be performed in accordance with the procedures specified in ASTM D8169/D8169M.

The Contractor shall install load cells and load test instrumentation in accordance with the bi-directional load cell supplier recommendations, instructions, and procedure manual(s), and in accordance with the drilled shaft installation plan approved by the Engineer.

The Contractor shall be responsible for coordinating with the load cell supplier to determine and/or verify all required equipment, materials, quantities, procedures, calibration data, and all other applicable items necessary to complete the load testing shown on the Plans.

The Contractor shall furnish, install and monitor load test instrumentation as shown on the Plans and as directed by the Engineer. .

A portable computer and electronic logging equipment shall be furnished to simultaneously monitor all instrumentation at time intervals designated by the Engineer.

The load cells, piping, instrumentation and other attachments shall be assembled and made ready for installation in accordance with the requirements of the bi-directional load cell supplier, unless otherwise specified herein or directed by the Engineer.

After completion of the load test to the satisfaction of the Engineer, and when authorized in writing by the Engineer, the Contractor shall flush all hydraulic fluid from the bi-directional load cells and hydraulic lines, and replace with cement grout in accordance with the approved Drilled Shaft Installation Plan. The Contractor shall also grout any voids remaining outside the load cells after completion of the load test.

X.6 TECHNIQUE SHAFTS

The Contractor shall demonstrate the adequacy of its methods, techniques and equipment by successfully constructing a technique shaft or shafts in accordance with the requirements shown on the Plans and these Specifications. The technique shaft(s) shall be positioned at the location(s) shown on the Plans or as directed by the Engineer, but not less than a clear distance of three drilled shaft diameters from the closest production shaft. The technique shaft(s) shall be drilled to the maximum diameter and maximum depth of any production drilled shaft shown in the Plans. Unless shown otherwise on the Plans, the technique shaft(s) shall be reinforced with the same reinforcement as the corresponding size production shaft, and shall also include CSL tubes as specified herein. Technique shaft(s) shall be completed and accepted by the Engineer prior to initiating installation of the load test shafts and foundation drilled shafts. Failure by the Contractor to demonstrate to the Engineer the adequacy of methods and equipment shall be reason for the Engineer to require alterations in equipment and/or method by the Contractor to eliminate unsatisfactory results. Any additional technique shaft(s) required to demonstrate the adequacy of altered methods or construction equipment shall be at the Contractor's expense. Once approval has been given by the Engineer to construct production drilled shafts, no changes will be permitted in the methods or equipment used to construct the satisfactory technique shaft(s) without the written approval of the Engineer.

COMMENTARY

The purpose of specifying a technique shaft (also sometimes referred to as trial, demonstration, or method shaft) or multiple technique shafts is twofold: first, to verify that the Contractor has the necessary expertise to complete the work successfully and, second, to determine if the proposed equipment and procedures are appropriate for the site conditions. Technique shafts are not necessary for every project; however, technique shafts should be considered for any of the following situations:

- *Site conditions are difficult or unusual for drilled shaft installation,*
- *The production drilled shafts are non-redundant foundation elements,*
- *Use of the wet method of drilled shaft construction, or*
- *For drilled shafts with diameter or depth greater than those commonly used in practice within the project area.*

The technique shaft(s) will be used by the Engineer to determine if the Contractor can: control dimensions and alignment of excavations within tolerance; install casing and remove temporary casing; seal the casing into impervious materials; control the size of the excavation under caving conditions by the use of a mineral or polymer slurry or by other means; properly clean the completed drilled shaft excavation; construct drilled shafts in open water areas; handle and install reinforcing cages; satisfactorily place concrete meeting the Specification requirements within the prescribed time limit; and to satisfactorily execute any other necessary construction operation.

When authorized in writing by the Engineer, the technique shaft(s) shall be cut off not less than 2.0 ft below finished grade and left in place. The disturbed areas at the sites of the technique shaft(s) shall be restored as nearly as practical to their original condition.

X.7 MEASUREMENT AND PAYMENT

X.7.1 Measurement

COMMENTARY

The cost for mobilization for drilled shaft construction should be included in a project-wide pay item for Mobilization rather than included in the unit prices for drilled shaft items or having a separate mobilization item limited to drilled shaft construction.

X.7.1.1 Drilled Shafts in Soil

Soil excavation for *drilled* shafts including haul *will* be measured by the lineal feet of *drilled* shaft excavated for each diameter. The lineal feet *will* be computed using the top of shaft soil excavation, as defined below, and the bottom elevation shown in the Plans, unless adjusted by the Engineer, less all rock excavation measured as specified in Article X.7.1.2.

COMMENTARY

An alternative to separate measurement and payment provisions for soil and rock excavation is to treat both soil and rock excavation as "Unclassified Excavation." See discussion in Chapter 14 of this Manual.

Except as otherwise specified, the top of shaft soil excavation *is* defined as the highest existing ground point within the *drilled* shaft diameter. For *drilled* shafts where the top of shaft is above the existing ground line and where the Plans show embankment fill placed above the existing ground line to the top of the *drilled* shaft and above, the top of shaft soil excavation *is* defined as the top of shaft concrete. Excavation through embankment fill placed above the top of shaft will not be included in the measurement.

X.7.1.2 Drilled Shafts in Rock

Rock excavation for *drilled* shafts including haul *will* be measured by the lineal feet of *drilled* shaft excavated for each diameter. The lineal feet *will* be computed using the *drilled* shaft diameter shown in the Plans, the top of the rock line, defined as the highest bedrock point within the *drilled* shaft diameter, and the bottom elevation shown in the Plans, unless adjusted by the Engineer.

COMMENTARY

Rock is defined as that consolidated mass of mineral material having an Unconfined Compressive Strength (UCS) in an intact sample of at least one sample of 1000 psi minimum. This definition falls between Class 1 and 2 of the relative rating system for rock classification outlined in Table 10.4.6.4-1 of the AASHTO LRFD Bridge Design Specifications.

Top of rock elevation for bidding purposes will be determined by the geologist's *or geotechnical engineer's* determination in the Contract Documents. Actual top of rock for payment purposes may differ from that shown in the Contract Documents based on the rock definition contained in *the commentary to Article X.7.1.2*.

COMMENTARY

The geologic determination for measurement purposes may be different from top of rock for design purposes to account for decomposed, weathered or shattered rock, *or variable rock surface*.

In some formations, such as pinnacle limestone, top of rock elevations may vary widely across the *drilled* shaft diameter, precluding the use of a single boring to accurately determine top of rock.

Some regional practices, such as the use of rig penetration rates to determine the top of rock, may need to be considered when developing rock pay quantities.

X.7.1.3 Obstruction Removal

Obstructions identified under Article X.4.4 will be measured per hour of time spent working on obstructions.

COMMENTARY

Alternatively, obstruction removal can be paid based on a force account basis.

The use of an hourly rate eliminates the necessity to maintain records of equipment on site and determine whether equipment was being used, on standby, or available for use elsewhere.

The hourly rate method does leave the process open to abuse through unbalanced bidding. *The hourly rate method requires monitoring of the time taken for obstruction removal, and agreement between the Contractor and Engineer on the level of effort and time required for obstruction removal.*

The alternative method of measuring and paying for obstruction removal includes payment on Force Account. While this eliminates the abuses of bid unbalancing, it does create a tremendous amount of administration to determine rates for equipment not commonly rated and record all equipment used or on standby. In addition, careful tracking of the equipment used and the effect of the obstruction removal on the equipment on site not used directly for obstruction removal, but subsequently idled by the obstruction event, will be needed.

Obstruction measurement and payment can be limited to unanticipated obstructions only. This method limits the incidence of obstructions and their payment. However, it places a heavy burden on the foundation report to accurately describe all known obstructions, and also encourages the Contractor to carry costly contingencies in its bid, and thereby potentially increasing bid prices unnecessarily.

X.7.1.4 *Technique* Shafts

Drilled shafts that are installed prior to installation of contract *drilled* shafts for the purpose of demonstrating to the Engineer the adequacy of the methods proposed *will* be measured per each for *technique* shafts installed successfully.

X.7.1.5 Exploration Holes

Exploration holes specified in the Contract *or directed* by the Engineer for purposes of confirming geotechnical properties of soil and rock and to determine the founding elevation of the proposed *drilled* shafts will be measured per lineal foot for Exploration Holes installed. Exploration holes may be drilled prior to *drilled* shaft excavation *or, for end bearing drilled shafts,* from the base of the *drilled shaft* excavation. The top elevation *will* be defined as *the* ground surface at time of exploration hole drilling. The bottom of *hole* elevation *will* be defined as the bottom of the exploration hole *approved by the Engineer*.

X.7.1.6 Permanent Casing – Furnishing and Placing

Furnishing and placing permanent casing *will* be measured by the number of linear feet of each diameter of required permanent casing installed, as specified in Article X.3.3. Upper limit of casing payment *will* be defined as the lower of:

- original ground, or
- base of footing

if excavated prior to *drilled* shaft installation. Lower limit *will* be the elevation indicated in the Contract Plans.

COMMENTARY

Alternatively, for payment purposes, both the top and bottom of permanent casing levels can be defined on the Plans. This approach is appropriate for cases where the limits of the permanent casing are defined by structural design requirements.

X.7.1.7 Load Tests

Load tests, *performed in accordance with these Specifications and accepted by the Engineer, will be* measured per each for *test shaft successfully installed in accordance with the dimensions and details shown on the Plans, and* carried successfully to the capacity specified or *to* shaft failure.

COMMENTARY

This item includes the work for both installing the load test shaft and performing the load test.

X.7.1.8 Cross-Hole Sonic Logging Casing

CSL access tube will be measured by the linear feet of tube furnished and installed.

COMMENTARY

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 *drilled* shaft cannot be accurately determined, replace Article X.7.1.8 with:

"CSL access tube will be measured by the linear foot of tube required based on the design depth shown in the Plans plus the length required to extend the *drilled* shaft reinforcement by set percentage of the length."

If CSL tubes are to be installed in all drilled shafts, the cost of the CSL tubes can be included in the pay item for Drilled Shaft Construction. This simplifies administration time for tracking CSL tube quantities.

X.7.1.9 . Drilled Shaft Construction

Concrete for *drilled* shaft *will* be measured by the cubic yards of concrete in place. The cubic yards *will* be computed using the *drilled* shaft diameter shown in the Plans, and the top and bottom elevations shown in the Plans, unless adjusted by the Engineer.

COMMENTARY

In cases where concrete is poured to limits of excavation (i.e. to the ground surface), serious consideration should given to combining bid items such as excavation, concrete placement, reinforcing steel placement (where rebar cages are constant in section throughout the entire shaft), *and CSL tubes.*

X.7.1.10 Reinforcing Steel

Steel reinforcing bar for *drilled* shaft *will* be measured by the computed weight in pounds of all reinforcing steel in place, as shown in the Plans. Bracing for steel reinforcing bar cages *will* be considered incidental to this item of work.

X.7.2 Payment

X.7.2.1 Drilled Shafts in Soil

Payment for the item "Soil Excavation for *Drilled* Shaft Including Haul," *will be* per lineal foot for each diameter, *including* all costs in connection with furnishing, mixing, placing, maintaining, containing, collecting, and disposing of all mineral, synthetic, and water slurry, and disposal of all excavated materials. Temporary casing required to complete *drilled* shaft excavation *is* included in this bid item.

X.7.2.2 Drilled Shafts in Rock

Payment for the item "Rock Excavation for *Drilled* Shaft Including Haul," *will be* per lineal foot for each diameter, *including* all costs in connection with disposal of spoil and associated water. Temporary casing, if necessary, is included in this bid item.

X.7.2.3 Obstruction Removal

Payment for *the item "Obstruction Removal in Drilled Shaft" will* be made for the changes in *drilled* shaft construction methods necessary to remove the obstruction, *and approved by the Engineer,* based on hours spent at Contract bid rates.

COMMENTARY

See commentary in Article X.4 and X.7.1.3 for additional guidance.

X.7.2.4 *Technique* Shafts

Payment for the item "Technique Drilled Shaft" will be paid on the basis of number of *technique drilled* shafts *shown on the Plans or* directed by the Engineer and installed successfully. Payment for *technique* drilled shafts *will* include mobilization, excavation and disposal of drill spoil, concrete, *CSL tubes,* and rebar, if necessary.

X.7.2.5 Exploration Holes

Payment for the item "Exploration Holes" installed *in accordance with the Contract Documents or* at the direction of the Engineer *will* be paid per lineal foot of exploration hole installed.

X.7.2.6 Permanent Casing – Furnishing and Placing

Payment for the item "Furnishing and Placing Permanent Casing for ____ Diameter *Drilled* Shaft," *will be* per linear foot

X.7.2.7 Load Tests

Payment for the item "Load Tests" *will* be per test *shaft successfully installed and successfully loaded to* failure or to the specified load.

X.7.2.8 Cross-Hole Sonic Logging Casing

Payment for the item "CSL Access Tube" will be per linear foot installed.

COMMENTARY

See Commentary for Articles X.7.1.8 and X.7.1.9.

If CSL testing is to be provided by the Contractor, then add the following Measurement and Payment specification:

- "Mobilization for CSL Test Paid per each Mobilization to test shafts.
- "CSL Test," per each shaft tested.

CSL test will be measured once per shaft tested.

X.7.2.9 Drilled Shaft *Concrete*

Payment for the item "Concrete for *Drilled* Shaft," *will* be per cubic yard

X.7.2.10 Reinforcing Steel

Payment of the item "Steel Reinforcing Bar for *Drilled* Shaft" *will* be per pound, including all costs in connection with furnishing and installing steel reinforcing bars, *stiffeners,* spacers and centralizers.

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APPENDIX E

SAMPLE DRILLED SHAFT INSPECTION CHECKLIST, FORMS, AND TABLES

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APPENDIX E
DRILLED SHAFT INSPECTOR'S CHECKLIST, FORMS, AND TABLES

Contractor and Equipment Arrive on Site	**YES**	**NO**	**N/A**
1. Has the Contractor submitted a Drilled Shaft Installation Plan?			
2. Has the Drilled Shaft Installation Plan been approved?			
3. Does Contractor have an approved concrete mix design?			
4. Has Contractor run the required Trial Mix and slump loss test for the concrete mix design?			
5. If concrete placement is estimated to take over two hours, has Contractor performed a satisfactory slump loss test for the extended time period?			
6. If Contractor proposed a mineral or polymer slurry, do they have an approved Slurry Management Plan?			
7. Have you attended pre-construction conference with the Engineer and Contractor for clarification of drilled shaft installation procedures and requirements?			
8. Is Contractor prepared to take soil samples or rock cores on the bottom of the shaft, if required in the Contract Documents?			
9. Has the Contractor met the requirements for protection of existing structures?			
10. Has the site preparation been completed as specified?			
11. Does Contractor have all the equipment and tools shown in the Drilled Shaft Installation Plan?			
12. If casing is to be used, is it the right size?			
13. If Contractor plans to use a slurry, do they have the proper equipment to mix it?			
14. Is the manufacturer's representative on site at the start of slurry work?			
15. If a slurry de-sander is required, does Contractor have it on site and operational?			
16. Does Contractor's tremie meet the requirements of the Specifications?			
17. Do you have all the drilled shaft forms that are needed during shaft construction?			
Technique Shaft			
18. Is the technique shaft positioned at the approved location?			
19. Has Contractor installed the technique shaft as specified?			
20. Did Contractor cut off the shaft below grade as specified?			
21. Does Contractor have approval for revised procedures and equipment identified during technique shaft installation?			
Shaft Excavation and Cleaning			
22. Is the shaft being constructed in the correct location and within tolerances?			
23. Does Contractor have a benchmark for determination of the proper elevations?			
24. If core holes are required, has Contractor taken them in accordance with the Specifications?			
25. If a core hole was performed, was a Rock Core form completed and did Contractor maintain a log?			
26. If Contractor is using slurry, did they perform tests and report results in accordance with the Specifications?			
27. Is the slurry level being properly maintained at the specified level?			
28. Are the proper number and types of tests being performed on the slurry?			
29. Are you filling out the Drilled Shaft Excavation forms?			
30. If permanent casing is used, does it meet requirements of Contract Documents?			

31. If temporary casing is used, does it meet the requirements of the Specifications?			
32. Is the shaft within allowable vertical alignment tolerance?			
33. Is the shaft of proper depth?			
34. Does the shaft excavation time meet the specified time limit?			
35. If over-reaming is required, was it performed in accordance with Specifications?			
36. Does the shaft bottom condition meet the requirements of the Specification?			
Reinforcing Cage			
37. Is the rebar the correct sizes and configured in accordance with the project plans?			
38. Is the rebar properly tied in accordance with the Specifications?			
39. Does Contractor have the proper spacers for the steel cage?			
40. Does Contractor have the proper number and spacing of spacers for the steel cage?			
41. If steel cage was spliced, was it done in accordance with Contract Documents?			
42. Is the steel cage secured from settling and from floating?			
43. Is the top of the steel cage at proper elevation in accordance with specified tolerance?			
Concrete Placement			
44. Prior to concrete placement, has the slurry (both manufactured and natural) been tested in accordance with the Specifications?			
45. Was the tremie pipe within specified maximum height above the shaft base at the start of concrete placement?			
46. Was a flap valve or "pig" used to separate concrete from slurry at the start of concrete placement?			
47. Was the discharge end of the tremie maintained at the specified minimum embedment in the concrete			
48. If free-fall placement (dry shaft construction only), was concrete place in accordance with the Specifications?			
49. Did concrete placement occur within the specified time limit?			
50. Are you filling out the Concrete Placement and Volume forms?			
51. Did Contractor overflow the shaft until good concrete flowed at the top of shaft?			
52. If required, was the casing removed in accordance with the Specifications?			
53. Were concrete acceptance tests performed as required?			
Post Installation			
54. Is all casing removed to the proper elevation in accordance with Specifications?			
55. If required, has Contractor complied with requirements for Integrity Testing?			
56. Is the shaft within the applicable construction tolerances?			
57. Have all Drilled Shaft inspection forms been completed?			
58. Have you documented the pay items?			
Notes/Comments:			

SAMPLE INSPECTOR'S "TOOLS"

CHECKLIST

Approved Job Information

- ☐ Project Plans & Specifications w/Revisions
- ☐ Special Provisions & Technical Special Provisions
- ☐ Drilled Shaft Installation Plan

References

- ☐ Standard Specifications
- ☐ Drilled Shaft Inspector's Manual (Local Department)
- ☐ Drilled Shaft Inspector's Qualification Course manual (NHI #132070

Testing Equipment

- ☐ Sampler
- ☐ Sand Content Testing Equipment
- ☐ Mud Density Test Equipment
- ☐ Viscosity Test Equipment

Blank Forms

- ☐ Drilled Shaft Soil/Rock Excavation Log
- ☐ Drilled Shaft Rock Core Log
- ☐ Drilled Shaft Inspection Log
- ☐ Concrete Placement Log
- ☐ Concrete Volume Form
- ☐ Drilled Shaft Log
- ☐ Drilled Shaft Construction & Pay Summary

Daily Essentials

- ☐ Hard Hat
- ☐ Boots
- ☐ Ear & Eye Protection
- ☐ Pen/Pencil (with spare)
- ☐ Pen/Pencil (with spare)
- ☐ 12' Tape (Preferably 25')
- ☐ 150' Tape
- ☐ Builders Square
- ☐ Life Jacket or reflective jacket
- ☐ Watch
- ☐ Calculator
- ☐ Camera
- ☐ Scale
- ☐ Level
- ☐ Weighted Tape (100')
- ☐ Plumb bob

SAMPLE

PLANS AND SPECIFICATIONS CHECKLIST			
The Inspector needs to be able to locate the following in the Plans and Specifications and be familiar with them before the job commences. **These documents should be with you at the job site and all times for reference.**			
YES	**NO**	**PLANS**	
		Revisions	
		Key Sheets	
		Construction Estimate Sheet	
		Plan/Profile Sheets	
		Traffic Control Plans	
		Drainage Plans	
		Utility Adjustments	
YES	**NO**	**STRUCTURES PLANS**	
		General Notes	
		Report of Core Borings	
		Foundation layout	
		Details	
		Bridge Hydraulic Sheet	
YES	**NO**	**SPECIFICATIONS**	
		Technical Special Provisions	
		Standard Specifications	
		Supplemental Specs	
		Drilled Shaft Installation Plan	

SAMPLE - SUMMARY OF DRILLED SHAFT INSTALLATION PLAN

a. Name of Drilled Shaft Superintendent_____ Experience_____ _____			FHWA Pub. IF-99-025 xxx.12 (a)
B. EQUIPMENT	MANUFACTURER	MODEL	SIZE
Drill Rig			
Crane			
Augers			
Casing			
Bailing Bucket			
Final Cleaning Equipment			
Desanding			
Slurry Pump			
Core Sampling Equipment			
Concrete Pump			

c. Sequence of Construction: How many crews___ How many shafts___ **c. Sequence of Shaft Construction:** Bents of Shaft groups

d. Details of Shaft Excavation Methods

e. Details of Slurry: Type _____ Methods to mix/circulate_____ Desand_____

Testing_____ Name of Lab_____

f. Details of method to clean Shafts after initial excavation:

g. Details of Shaft reinforcement:

h. Details of Concrete placement procedures: Concrete or Pump tremie_____ Initial placement_____

Raising during placement_____ Overfilling shaft_____

Provisions to ensure final shaft Cutoff Elevation:

i. Details of casing Removal:

1. Required Submittals: Shaft Drawings _____ Concrete Mix Design_____

2. Details of Load Test: Equipment_____ Procedure_____ Calibration for Jacks or Leadoel's _____

3. Prevention of Displacement of Casing/Shafts during Placement
Compaction of Fill_____ Method_____ Equpiment_____

4. Environmental control procedures to prevent loss of slurry or concrete into waterways:

5. Other information:

SAMPLE

DRILLED SHAFT SOILD EXCAVATION LOG (ENGLISH/METRIC)

Project Name				Page	of
Project No				Pier No.	
Contractor				Shaft No.	
Inspected By		Date		Station	
Approved By		Date		Offset	

Casting Information

ID	OD	Top Elev.	Length	Bot. Elv.

Soil Auger Diam.	
Gmd. Suif Elev.	
Water Table Elev.	
Reference Elev.	
Drilling Mud	

Notes

Depth	Elev.	Time		Soil Description & Notes
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	

SAMPLE
DRILLED SHAFT ROCK EXCAVATION LOG
(ENGLISH/METRIC)

Project Name				Page	of
FIN Project No.				Pier No.	
Contractor				Shaft No.	
Inspected By		Date		Station	
Approved By		Date		Offset	

Rock Auger Diameter		Reference Elev.	
Core Tool Diameter		Med. Top of Rock Elev.	
Overream Tool Diameter		Med Shaft Bottom Elev.	

Notes

Depth	Elev	Rpm	Time	Rock Description & Notes

SAMPLE

	GEOTECHNICAL ENGINEERING BUREAU **DRLLED SHAFT IN SOIL – FIELD RECORD**	
	STRUCTURE	
	SHAFT NUMBER	DATE

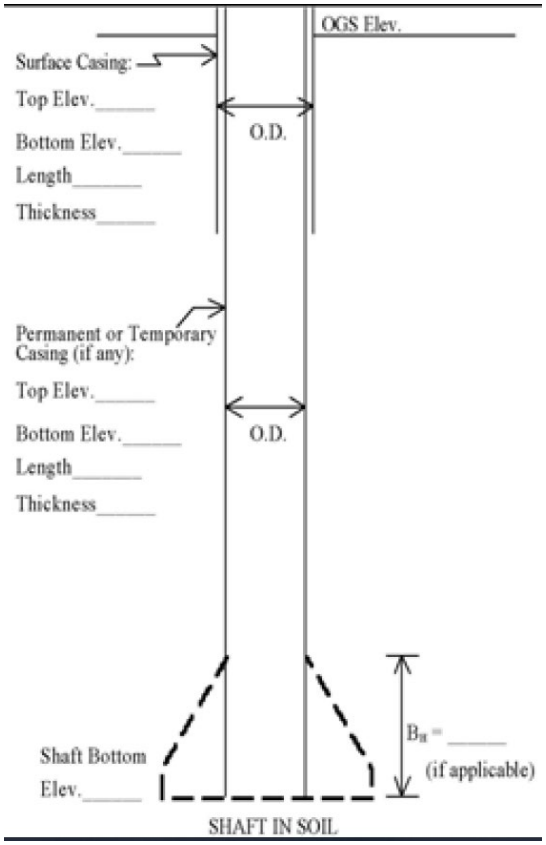
PROJECT STAMP

Date Excavation Started: Finished:

Date Bottom Observed:

Date Concrete Placed:

SHAFT DETAIL	**DESIGN**	**AS-BUILT**
Station		
Offset		
Top Elevation		
Bottom Elevation		
Shaft Diameter		
Shaft Length		
Bell Diameter (if appl.)		
Bell Height B_H (if appl.)		
Plumbness		



Design Capacity:

Observed Groundwater Elevation:

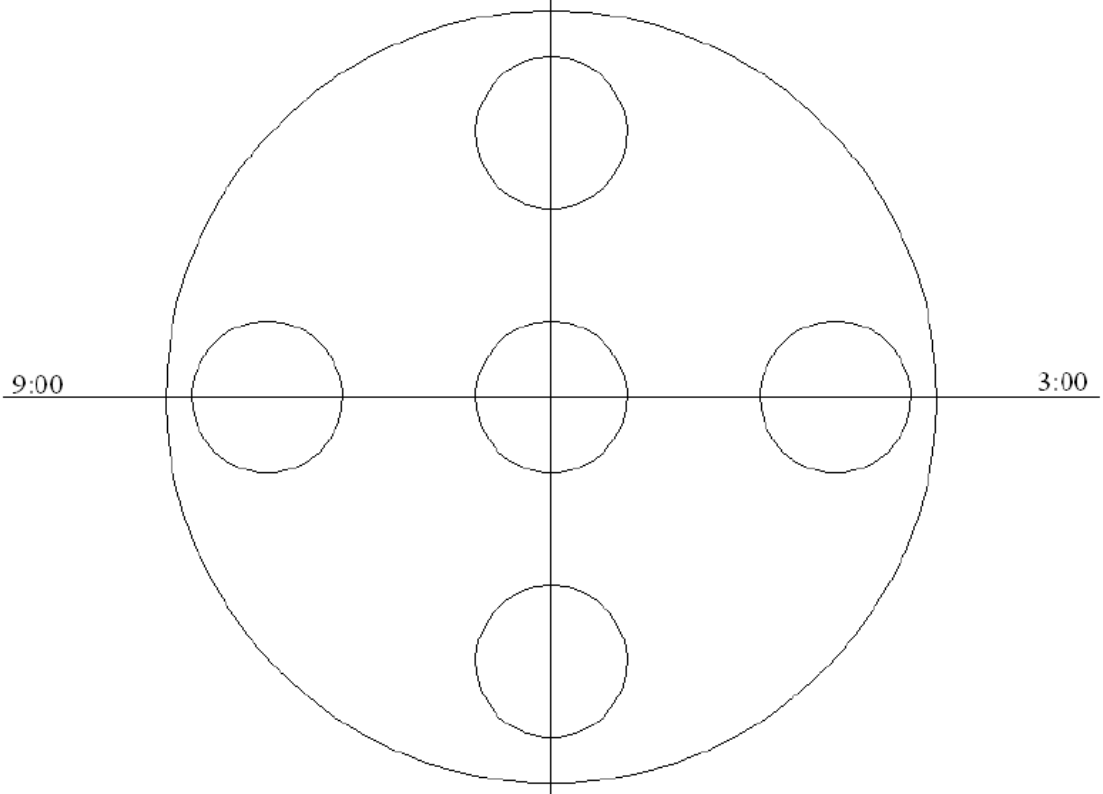
Remarks:

Date	Time	Depth	Soil or Rock Description	Tool	Observations

SAMPLE
DRILLED SHAFT INSPECTION
(ENGLISH/METRIC)

Project Name		Page	of
FIN Project No.		Pier No.	
Contractor		Shaft No.	
Inspected By	Date	Station	
Approved By	Date	Offset	

Inspection By: _____ Visual
_____ Sounding
Time Stated _____
Time Finished _____



Comments

_____
_____

Recommendations

_____
_____

Given To: _____ Verbal/Written

Results: _____ Satisfactory

Unsatisfactory By: _____ Time: _____ Date: _____

SAMPLE
DRILLED SHAFT CONCRETE PLACEMENT LOG
(ENGLISH/METRIC)

Project Name		Page	of
FIN Project No.		Pier No.	
Contractor		Shaft No.	
Inspected By	Date	Station	
Approved By	Date	Offset	

Placement Method	Tremie	Volume in Lines	#	ID	Length	Volume
Deairing Method	Pumped					
	Relief Valve					
	Tremie Plug					
	Tremie Cap					
		Total Volume in Lines				

Reference Elev. _____

Shaft Top Elev. _____

Top of Rock Elev. _____ Depth To Water Inside _____ OD Casing At Start _____

Shaft Bottom Elev. Rebar Cage Top Elev. At Start At Finish

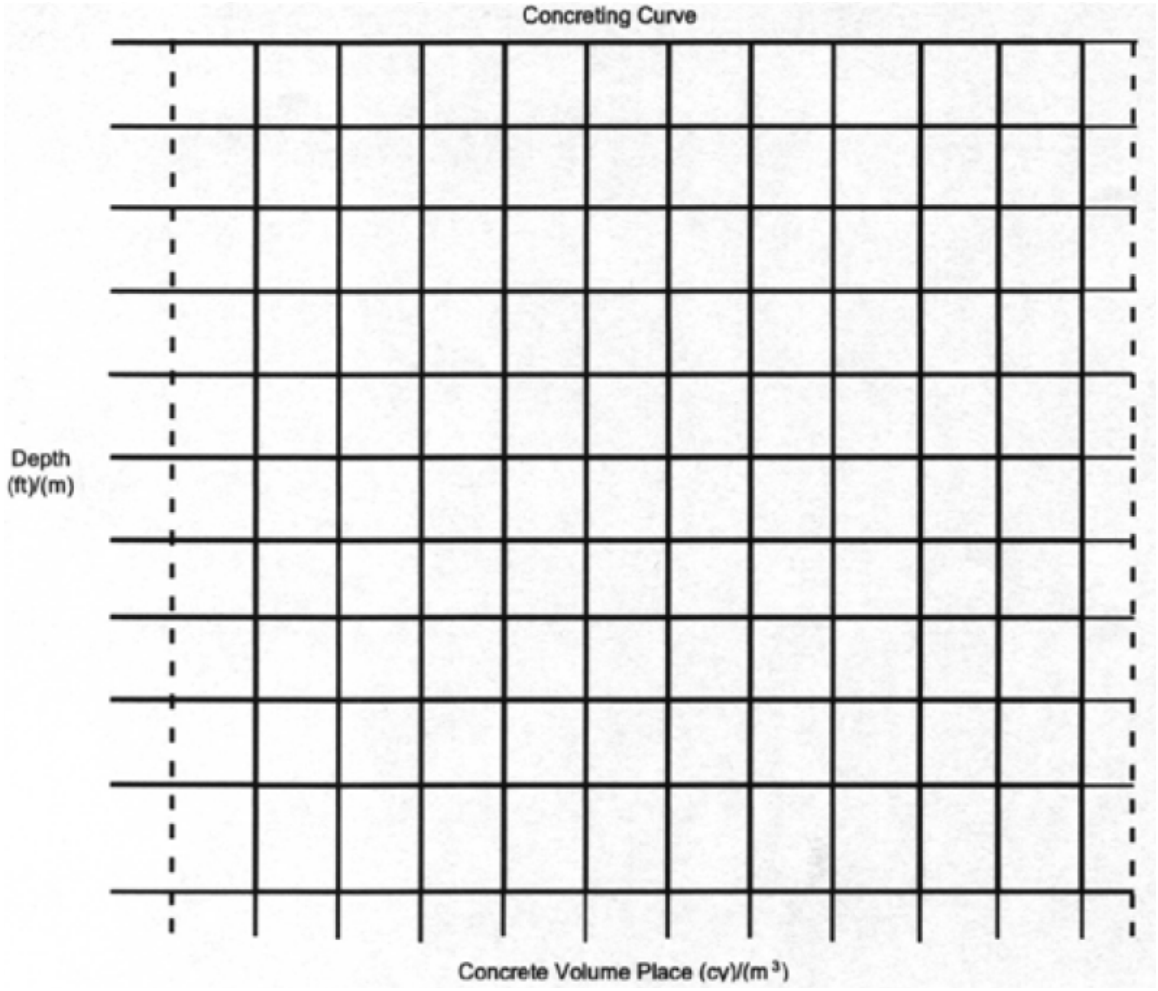
Truck No.	Concrete Volume	Arrival Time	Start Time	Finish Time	Tremie Depth	Depth To Concrete	Notes

_____ Concrete Volume Delivered Placement Time (Casing Removed) _____

Details	OD	Top Elev.	Bot. Elev.	Start	Finish	Rebar Cage Centered	
Casing						Concrete Finish	
Removal							
Notes							

SAMPLE
DRILLED SHAFT CONCRETE VOLUMES
(ENGLISH/METRIC)

Project Name		Page	of
FIN Project No.		Pier No.	
Contractor		Shaft No.	
Inspected By	Date	Station	
Approved By	Date	Offset	



Volume Delivered	VD	_____	cy/m^3
Volume in Lines	VL	_____	cy/m^3
Wastage	VW	_____	cy/m^3
Volume Placed = VD-VL-VW =	VP	_____	cy/m^3
Theoretical Vol.	VT	_____	cy/m^3
Overpour (VP-VT)	OP	_____	cy/m^3

Lineal Volume of Rebar, Telltales, etc. AR

(#bars)(As)+(#)(At)=( )(si) =	_____	s
(#bars)(As)+(#)(At)=( )(m^2)+( )(m^2) =	_____	m^2
Rock Socket Length (RSL)	_____	ft/m
Est. Rock Socket Volume From Curve (VRS)	_____	cy/m^3
Act. Rock Socket Volume VRS' = VRS+(AR)(RSL)-VI		
(cy)+(si)(ft)/(3888)-(cy) =	_____	cy
(m^3)+(m^2)(m)-(m^3) =	_____	m^3
Avg. Rock Socket Dia. = SQRT[VRS'/(0.7854)(RSL)]		
SQRT[(4950)(cy)/(ft)] =	_____	in
SQRT[(1.273)(m^3)/(m)] =	_____	m

DRILLED SHAFT CONSTRUCTION & PAY SUMMARY
(ENGLISH/METRIC)

CONSTRUCTION
02/01

Project Name		Page	of
FIN Project No.		Pier No.	
Contractor		Shaft No.	
Inspected By	Date	Station	
Approved By	Date	Offset	

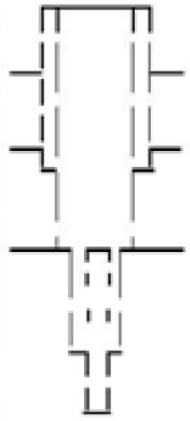
Type of Construction

_____ Dry _____ Wet/Drill mud only _____ Wet/Casing only _____ Wet/Drill mud & casing

Casing _____ None _____ Removed _____ Permanent

Notes:

Construction Activity	Time	Date mm/dd/yy
Set Casing		
Begin Excavation (Below Casing)		
Beg. Soil Excav.		
Fin. Soil Excav.		
Beg. Rock Excav.		
Fin Rock Excav.		
Rock Core Taken		
Overream		
Init. Inspection		
Beg. Concrete Pl.		
Fi. Concrete Pl.		
Constr. Complete		



Shaft Details	Abbrev.	Plan	Built
Shaft Top El.	ST		
Casing Top El.	CT		
Water Table El.	WT		
Perm Casing ID	CID		
Per Casing OD	COD		
Ground Surf. El.	GS		
Casing Bot El.	CB		
Shaft Diameter	SD		
Top of Rock El.	RT		
Rk. Core Top El.	RCT		
Rock Core Dia.	RCD		
Rock Socket Dia.	RD		
Overream Top El.	OT		
Overream Bot. El.	OB		
Shaft Bottom El.	SB		
Rk. Core Bot. El.	RCB		

Detail	Abbrev.	Length Provided (ft)/(m)	Length Adjustment (ft)/(m)	Pay Length (ft)/(m)	Notes
Permanent Casing	CT-CB				
Soil Excavation	GS-RT				
Rock Excavation Extra Depth Excav. SBplan=SBbuilt	RT-SB				
Overream length	OT-OB				
Drilled Shaft Length	ST-SB				
Rock Core Length	RCT-RCB				

Adjustments

Soil Excavation = (GS-RT)(SDplan-SDbuilt)/SDplan = (-) = _____
SD (built<plan)

Rock Excavation = (RT-SB)(RDplan-RDbuilt)/RDplan = (-) = _____
RD (built<plan)

Drilled Shaft Length = Excavation Adjustments + Any Other Adjustments

FHL-E-1

DRILLED SHAFT SOIL EXCAVATION LOG

Project Name		Pier No.	ABUT-1
Project No.	86180-3522	Shaft No.	3
Contractor		Station	32+60.00
Inspected By	Date 3-28-00	Offset (m)	7.26 RIGHT
Approved By	Date		

Casing Information

ID (mm)	OD (mm)	Top Elev. (m)	Length (m)	Bot. Elev. (m)
1200	1220	+4.30	8.40	-9.10
		19.70	18.99	-19.29

Soil Bucket Dia. (mm)	1160
Mud Line/Ground Elev.	+2.60
Water Table Elev. (m)	
Reference Elev. (m)	+9.00
Drilling Mud	N/A

Notes 2.39 OFFSET.

Date	Elevation (m)	Time		Soil Description and Notes
3-28-00		2:38	in	
46" INCLUDING BUCKET	+1.30	2:40	out	TAN LIMESTONE FILL
		2:41	in	
	+0.62	2:43	out	SAME
		2:49	in	
	-0.20	2:45	out	SAME
		2:46	in	
	-0.53	2:48	out	TAN & GRAY SANDY LIMESTONE FILL
		2:49	in	
	-1.23	2:51	out	BLACK SILTY SAND (ORGANIC)
		2:52	in	
	-1.94	2:54	out	SAME
		2:55	in	
	-2.60	2:57	out	GRAY FINE SAND & SHELL
		2:58	in	
	-3.16	3:00	out	GRAY & TAN FINE SAND & SHELL
		3:02	in	
	-4.40	5:09	out	TAN SAND WITH CEMENTED LAYERS
3-30-00		9:47	in	
	-6.10	9:50	out	GRAY FINE SAND
		9:51	in	
	-6.68	9:55	out	TAN & GRAY FINE SAND & SHELL
		9:57	in	
	-7.59	10:01	out	GRAY

Approved _____
Soils Engineer

drlexcom

DRILLED SHAFT INSPECTION

Project Name _____ Pier No. ABUT-1
Project No. 86180-3522 Shaft No. 3
Contractor _____ Station 32160.00
Inspected By _____ Date 4-4-00 Offset (m) 7.26 RT
Approved By _____ Date _____

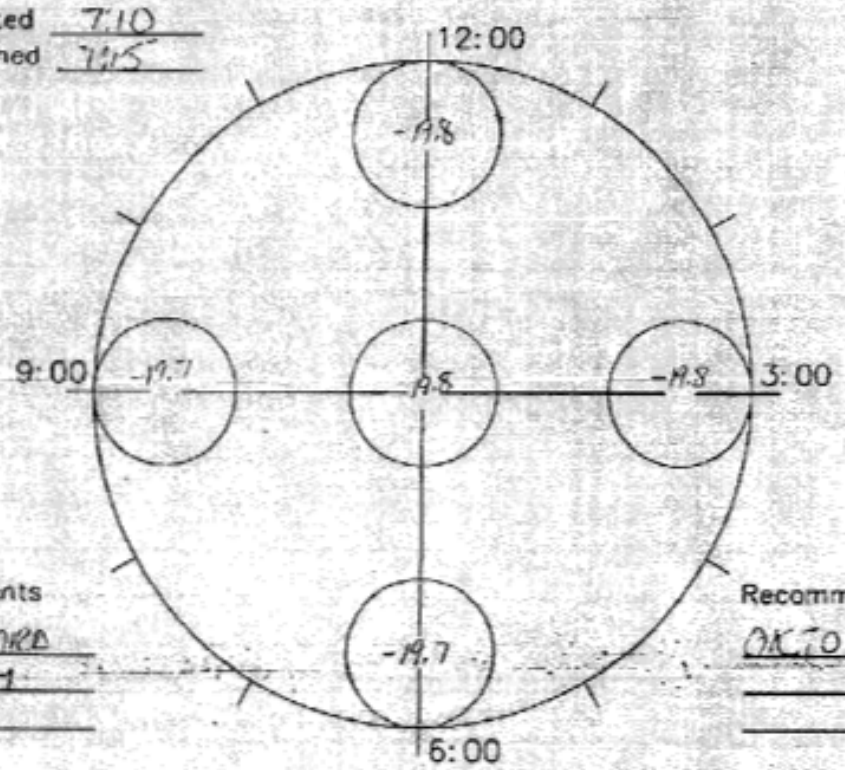
Type of Drilling Fluid WATER
Drilling Fluid Check 1270 KG/M³
Bottom Clean-out Method BAILER
Time/Date Final Clean-out 6:40AM 4-4-00
Shaft Bottom Elev. (m) -19.8
Est. Shaft Bottom Dia. (mm) 1220

Shaft Plumbness Check ✓
Rebar Cage:
Proper - Vert. Bars ✓
Proper - Horiz. Bars ✓
Side Stand-offs ✓
Bottom Stand-offs ✓
Epoxy Condition N/A
Ties and Connections ✓

Inspected by ✓ Visual (S.I.O.)
✓ Sounding
___ Other

Job North at 12:00

Time Started 7:10
Time Finished 7:15



Comments
GOOD FIRM BOTTOM

Recommendations
OK TO POUR

esults: ✓ Satisfactory ___ Unsatisfactory
Given to _____ By _____
(Verbal)/Written
Time 7:20AM Date 4-4-00

Approved _____
Soils Engineer

DRILLED SHAFT CONCRETE PLACEMENT LOG

Project Name _____ Page ___ of ___
Project No. 86180-3522 Pier No. ABUT-1
Contractor 1 Shaft No. 3
Inspected By _____ Date 4-4-00 Station 32+60.00
Approved By _____ Date _____ Offset (m) 7.26 LT

Placement Method: ✓ Tremie / Pumped
Deairing Method: Relief Valve / ✓ Tremie Plug / Tremie Cap

Volume in Lines	#	ID	Length	Volume
Total Volume in Lines			Σ	0.764

Reference Elev. +4.00
Shaft Top Elev. +3.00
Top of Rock Elev. -Ka.00
Shaft Bottom Elev. -14.8

Depth To Water Inside 2M. OD Casing At Start 1220
Rebar Cage Top Elev. At Start +3.85 At Finish +3.77 +3.82

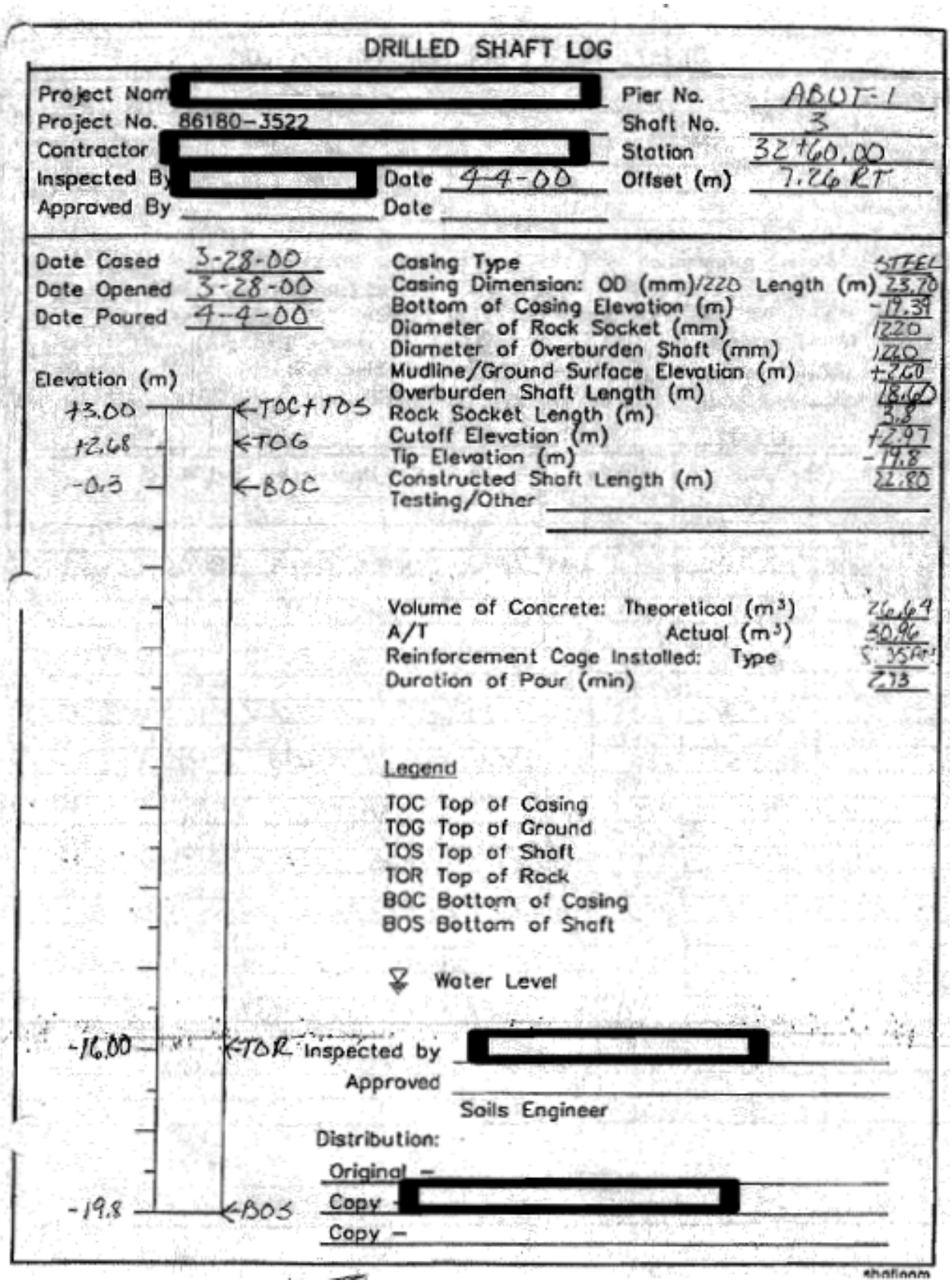
Truck No.	Concrete Volume	Arrival Time	Start Time	Finish Time	Tremie Depth	Depth To Concrete		Notes
1	6.88M³	8:06	8:34	8:42	22.0M	19.9	-15.1	1ST TRUCK BATCHED AT 7:5
2	6.88M³	9:22	9:29	9:33	18.0M	13.7	-9.9	PULLED OUT 7.5M CASING
3	6.88M³	10:43	10:48	11:03	12.0M	8.3	-4.3	CONCRETE AT 9.70M.
4	6.88M³	10:59	11:19	11:24	10.0M	2.0	+2.0	PULLED OUT 13M CASING
5	6.88M³	12:25	1:03	1:07	7.0M	TOC.		CONCRETE AT +1.00M

34.4M³ Concrete Volume Delivered — Placement Time (Casing Removed) 4HR 55MIN.

	OD	Top Elev.	Bot. Elev.	Start	Finish
Casing	1220	+4.00	-12.76	9:52	10:05
Removal	1220	+3.10	-0.3	11:26	11:44

Rebar Cage Centered OK
Concrete Finished 1:10 P.M.

Notes _____



DRILLED SHAFT LOG

Project Nam [redacted]
Project No. 86180-3522
Contractor [redacted]
Inspected By [redacted] Date 4-4-00
Approved By _____ Date _____

Pier No. ABUT-1
Shaft No. 3
Station 32+60.00
Offset (m) 7.26 RT

Date Cased 3-28-00
Date Opened 3-28-00
Date Poured 4-4-00

Casing Type STEEL
Casing Dimension: OD (mm)/220 Length (m) 23.70
Bottom of Casing Elevation (m) -19.39
Diameter of Rock Socket (mm) 1220
Diameter of Overburden Shaft (mm) 1140
Mudline/Ground Surface Elevation (m) +2.60
Overburden Shaft Length (m) 18.60
Rock Socket Length (m) 3.8
Cutoff Elevation (m) +2.97
Tip Elevation (m) -19.8
Constructed Shaft Length (m) 22.80
Testing/Other _____

Elevation (m)
+3.00 ←TOC+TOS
+2.68 ←TOG
-0.3 ←BOC
-16.00 ←TOR
-19.8 ←BOS

Volume of Concrete: Theoretical (m^3) 26.69
A/T Actual (m^3) 30.96
Reinforcement Cage Installed: Type S 35A
Duration of Pour (min) 273

Legend

TOC Top of Casing
TOG Top of Ground
TOS Top of Shaft
TOR Top of Rock
BOC Bottom of Casing
BOS Bottom of Shaft

Water Level

Inspected by [redacted]
Approved _____
Soils Engineer

Distribution:
Original –
Copy – [redacted]
Copy –

APPENDIX F

DRILLED SHAFT ACCEPTANCE: EXAMPLE FORMS AND MITIGATION PLANS

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APPENDIX F
DRILLED SHAFT ACCEPTANCE:
EXAMPLE FORMS AND MITIGATION PLANS

Example forms are provided for evaluation of drilled shafts following installation and nondestructive testing. Two forms are shown, one based on gamma-gamma logging (GGL) and the second based on cross-hole sonic log testing (CSL). These forms were developed by the California Department of Transportation (Caltrans) and are available on the Caltrans website. The process of acceptance for drilled shafts (CIDH Piles) which exhibit GGL or CSL anomalies requires evaluation and completion of the form by four engineering entities: foundation testing, geotechnical, structural, and corrosion engineers.

The forms are followed by a document entitled "Standard Drilled Shaft Anomaly Mitigation Plan." This document was developed by the International Association of Foundation Drilling: ADSC, and grew out of a joint effort by the ADSC West Coast Chapter and Caltrans to establish a process for evaluation and, if needed, mitigation of drilled shafts.

Information provided in this appendix is intended as a resource for public agencies seeking to develop quality control and quality assurance procedures as part of the drilled shaft design and construction process.

3-7 PILE DESIGN DATA FORM (GGL)

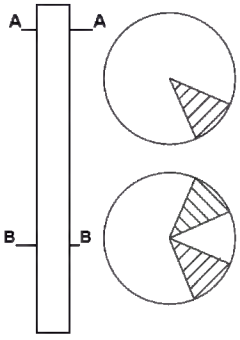
1 Foundation Testing — Name: **GS-FTB** Phone: Date:

Anomaly Overview

Testing Performed **X** GGL ☐ CSL

Shaft Diameter: 8 ft

Cutoff Elev: -29 ft



Section A-A

Elev.: -32 ft to -34 ft

Up to 12.5% Affected

Section B-B

Elev.: -65 ft to -67 ft

Up to 25% Affected

Tip Elev.: -113 ft

Anomaly Description

Section A-A: Anomaly was detected in one (1) GGL inspection tube. May affect up to 12.5% of Shaft cross-section at this location.

Section B-B: Anomalies were detected in two (2) GGL inspection tubes. May affect up to 25% of Shaft cross-section at this location.

2 Geotechnical — Name: **GS** Phone: Date:

Required Nominal Resistance of Shaft (per contract plans)

Compression:_____kips Tension:_____kips

Lowest Estimated Groundwater Elevation: _____

Remaining Required Nominal Resistance To Be Developed Below Each Anomalous Section:

Section A-A: **Compression**_____**Tension**_____**kips**

Soil and/or Rock Type: _____

Shaft is geotechnically Acceptable Unacceptable

Section B-B: **Compression**_____**Tension**_____**kips**

Soil and/or Rock Type: _____

Shaft is geotechnically Acceptable Unacceptable

Comments:_____

3 Structural — Name: **SD** Phone: Date:

As-Designed Capacity of Shaft

Section A-A: Shear: 1355 kips Moment: 23224 kip-ft

Section B-B: Shear: 1355 kips Moment: 23224 kip-ft

Maximum Demand of Shaft at Section A-A

Shear: 813 kips Moment: 21500 kip-ft

Shaft is structurally ☐ Acceptable X Unacceptable

Maximum Demand of Shaft at Section B-B

Shear: 913 kips Moment: 13700 kip-ft

Shaft is structurally X Acceptable Unacceptable

Comments:_____

4 Corrosion — Name: **METS** Phone: Date

Consideration is ☐ Required ☐ Not required

For anomalies between the top of pile and 3 feet below the lowest estimated groundweter level at the site, corrosion results listed in the Geotechnical report are used to assess the need for repair. For situations where results are not available, soil samples mey be obteined adjecent to the anomaly end tested in accordance with California Test (CT) 643 (Parts 2, 3 and 4) and if necessary, CT 417 and CT 422 to determine soil corrosivity. For anomelies outside these limits, and where no stray current source can be identified, or for non-corrosive soil conditions, no consideration of corrosion potentiel is required.

Corrosion Potential at Section A-A: _____

Corrosion Potential at Section B-B: _____

5 Construction Considering parts 2-4 of this form, — Structure Rep.: **SC** Phone: Date:

Sec. A-A is: ☐ Acceptable with Administrative Deduction ☐ Unacceptable, Mitigation is Required

Sec. B-B is: ☐ Acceptable with Administrative Deduction ☐ Unacceptable, Mitigation is Required

Bridge:	Bridge No.:	Abt./Bent:
Dist-Co.-Route:	EA:	Pile:
Structure Rep.:	Phone:	Fax:

3-7 PILE DESIGN DATA FORM (CSL)

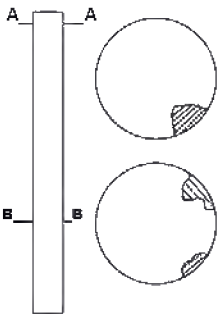
1 Foundation Testing Name: GS-FTB Phone: Date:

Anomaly Overview

Testing Performed X GGL X CSL

Shaft Diameter: 8 ft

Cutoff Elev: -29 ft



Section A-A

Elev.: -32 to -34 ft

Up to 9 % Affected

Section B-B

Elev.: -65 to -67 ft

Up to 8 % Affected

Tip Elev.: -113 ft

Anomaly Description

Section A-A: Anomaly was detected in one (1) GGL inspection pipe and four (4) CSL pipe pairs. May affect up to 9% of Shaft cross-section at this location.

Section B-B: Anomalies were detected in two (2) GGL inspection pipes and three (3) CSL pipe pairs. May affect up to 8% of Shaft cross-section at this location.

2 Geotechnical Name: GS Phone: Date:

Required Nominal Resistance of Shaft (per contract plans)

Compression: kips Tension: kips

Lowest Estimated Groundwater Elevation: _____

Remaining Required Nominal Resistance To Be Developed Below Each Anomalous Section:

Section A-A: Compression: _____ **Tension:** _____ **kips**

Soil and/or Rock Type: _____

Shaft is geotechnically Acceptable Unacceptable

Section A-A: Compression: _____ **Tension:** _____ **kips**

Soil and/or Rock Type: _____

Shaft is geotechnically Acceptable Unacceptable

Comments: _____

3 Structural Name: SD Phone: Date:

As-Designed Capacity of Shaft

Section A-A: Shear: 1355 kips Moment: 23224 kip-ft

Section B-B: Shear: 1355 kips Moment: 23224 kip-ft

Maximum Demand of Shaft at Section A-A

Shear: 813 kips Moment: 21500 k-ft

Shaft is structurally ☐ Acceptable **X** Unacceptable

Maximum Demand of Shaft at Section B-B

Shear: 913 kips Moment: 13700 k-ft

Shaft is structurally **X** Acceptable Unacceptable

Comments: _____

4 Corrosion Name: METS Phone: Date: Consideration is ☐ Required ☐ Not required

For anomalies between the top of pile and 3 feet below the lowest estimated ground water level at the site, corrosion results listed in the Geotechnical report are used to assess the need for repair. For situations where results are not available, soil samples may be obtained adjacent to the anomaly and tested in accordance with California Test (CT) 643 (Parts 2, 3 and 4) and if necessary, CT 417 and CT 422 to determine soil corrosivity. For anomalies outside these limits, and where no stray current source can be identified, or for non-corrosive soil conditions, no consideration of corrosion potential is required.

Corrosion Potential at Section A-A: _____

Corrosion Potential at Section B-B: _____

5 Construction Considering parts 2-4 of this form, Structure Rep.: SC Phone: Date:

Sec. A-A is: ☐ Acceptable with Administrative Deduction ☐ Unacceptable, Mitigation is Required

Sec. B-B is: ☐ Acceptable with Administrative Deduction ☐ Unacceptable, Mitigation is Required

Bridge:	Bridge No.:	Abt./Bent:
Dist-Co.-Route:	EA:	Pile:
Structure Rep.:	Phone:	Fax:



STANDARD DRILLED SHAFT ANOMALY MITIGATION PLAN

ADSC Drilled Shaft Committee
NOVEMBER 2014

INTRODUCTION

As Drilled Shaft Subcontractors, one of our main goals in the execution of our work is to install a quality product. The owner will typically include a Non-Destructive Testing (NDT) requirement in the contract specifications in order to verify the contractors' installation techniques. Typical NDT includes, but is not limited to: Gamma-Gamma Logging (GGL), Crosshole-Sonic Logging, (CSL), and Thermal Integrity Profiling (TIP). GGL is typically performed in 2" pvc pipes that are tied to the reinforcing cage prior to placement into the Shaft. CSL is typically performed in 2" steel pipe (although pvc can be used) that are tied to the reinforcing cage prior to placement into the Shaft. TIP can be performed in pipes that are tied to the reinforcing cage prior to placement into the Shaft or a wire with thermal couples is attached at equal intervals around the reinforcing cage prior to placement into the Shaft.

No matter how diligent our crews are in drilling, cleaning and placing concrete; occasionally an anomaly (defined as: a deviation from the norm, an irregularity) will be discovered during the Non-destructive testing (typically performed by the owner / public agency). When an anomaly is discovered, the engineer of record is requested to determine if the anomaly will require a repair or whether it is acceptable as-is.

Most of the time, the work performed by the Drilled Shaft Contractor is on the Critical Path of the project schedule. When an anomaly is required to be repaired, the time it takes to put a plan together and get it approved can greatly affect the schedule.

The intent of this "Standard Drilled Shaft Anomaly Mitigation Plan" is to have a pre-approved plan that can be used for several types of "typical" anomalies prior to the start of work so that any repairs can performed quickly and with little impact to the project schedule.

The following recommendations / plans are based on the "ADSC Standard Mitigation Plan" that was written by the ADSC West Coast Chapter and reviewed and approved by Caltran's (California Department of Transportation) Foundation Testing Branch and implemented for use in 2007. They have been reviewed and edited by the ADSC-IAFD Drilled Shaft Committee to be used in conjunction with Non-Destructive Testing Methods listed above.

Since NDT can be performed in either PVC or Steel Pipes, access to an anomaly can be gained by the high-pressure removal of the pvc pipe by water jetting or by drilling / coring down to the anomalous zone. When steel pipes are used, it is recommended to

install pvc caps rather than steel caps on the bottom of the pipes to allow for easy removal should grouting at the tip of the pile be required.

This Standard Drilled Shaft Anomaly Mitigation Plan consists of two Plans: Standard Mitigation Plan "A" which contains typical procedures for a "basic repair" of anomalies, and Standard Mitigation Plan "B" which contains typical procedures for mitigation by replacement and/or permeation grouting methods.

To address the wide range of anomalies that may be candidates for grouting repair, Standard Mitigation Plan "B" is broad in scope and contains provisions that will not be applicable to all anomalies. Although the generic nature of the plan may lead to some inefficiency in mitigation operations, the intention of the standard plan is to expedite the acceptance of typical mitigation plans by providing a template for formal submission to the owner. The submittal of an anomaly mitigation plan can be made job specific by the attachment of a cover letter which addresses the specific site conditions and contract requirements.

These standard mitigation plans do not address specific anomalies. Thus, the plans do not contain some of the components of a typical mitigation plan, such as a description of the subject anomalies and a review of project documents.

These standard plans do not purport to address safety concerns associated with their use. It is the responsibility of the contractor to establish appropriate health and safety practices and to determine the applicability of regulatory limitations prior to performing the work.

STANDARD MITIGATION PLAN "A" - BASIC REPAIR

OVERVIEW OF MITIGATION PLAN "A"

Standard mitigation plan "A" describes a typical procedure for basic repair. Basic repair involves the mechanical removal and replacement of concrete within an anomalous zone. Basic repair is typically performed by hand; thus, the anomalies to be repaired must be accessible, or the anomalies must be accessed by excavation. Basic repair performed near the top of a pile is also known as "simple" repair. Restoration of earthen materials may be required if the anomalies to be repaired are not immediately accessible.

MITIGATION PLAN "A"

A basic repair involves mechanical removal and replacement of concrete from anomalies accessed from the ground surface or by excavation. Mechanical removal is performed using a chipping gun or similar means under the observation and direction of the inspector / engineer.

A. Excavation

1. Excavate alongside the drilled shaft in the vicinity of the designated inspection tube(s) to a depth of one foot below the identified anomaly to provide access. Shoring plans, confined space plans, and provisions for replacement of earthen materials disturbed by excavation shall be submitted as appropriate.

B. Removal of Deleterious Material

1. After excavation and exposure of the anomaly, all visually deleterious or questionable material will be removed. Mechanical removal will "chase" all inclusions or compromised concrete until competent concrete is encountered.

2. If the surface of the Drilled shaft shows apparently competent concrete, a small section in the center of the identified anomaly will be chipped a minimum depth of one inch to demonstrate that the surface manifestation is consistent with concrete below the pile surface.

C. Inconclusive Results

1. If visual inspection is inconclusive, a 2 or 3-inch diameter core sample can be obtained from the anomalous zone for additional visual inspection and/or strength testing. The core should extend 6 to 12-inches beyond the anomalous zone or as approved by the owner. Strength testing shall be performed in accordance with standard testing procedures.

2. If visual inspection or the results of compressive strength testing indicate that the concrete is not acceptable, the unacceptable concrete shall be mechanically removed to the extent determined by the inspector.

3. If the results indicate that the concrete is acceptable as-is, the contractor should be compensated for the investigative work performed.

D. Verification of Results by Engineer

1. After the Contractor has removed all material that is visibly compromised or questionable, the Engineer will visually and physically inspect the effectiveness of the removal operation. If the concrete is deemed acceptable, the removal will be terminated and approved. If additional questionable material is identified, the Contractor shall remove this material and request that the Engineer re-inspect.

E. Replacement

1. After removal of unacceptable concrete and questionable material, forms shall be constructed as necessary, and the specified concrete mix shall be placed in the repair area, or alternatively, at the acceptance of the engineer/owner, the area can be poured back with the follow-on concrete work (column pour or footing pour)

2. After the concrete has cured, forms shall be removed.

F. Restoration of Earthen Materials

1. Earthen materials shall be replaced as approved by the owner. Where not otherwise designated, earthen materials shall be replaced using the excavated soil and compacted to a relative density that approximates the undisturbed, in-situ density of adjacent earthen materials. Two-sack sand slurry may be used if the Engineer of Record indicates that this will not adversely affect the lateral stiffness of the pile.

G. Reporting

1. Upon completion of the mitigation procedure, a mitigation report shall be submitted stating what repair work was performed and whether the repair work conformed with the mitigation plan. Any deviations from the mitigation plan must be stated in the report.

STANDARD MITIGATION PLAN "B" - GROUTING REPAIR

OVERVIEW OF MITIGATION PLAN "B"

Standard mitigation plan "B" describes typical procedures for grouting repair. Pile design data, construction details, subsurface conditions, and the results of conformance testing are considered in the development of a grouting mitigation plan. The formation conditions and extent of communication often cannot be completely characterized by the results of water flow testing and thus may be largely unknown at the start of grouting. Thus, provisions for both replacement grouting and permeation grouting are presented in standard mitigation plan "B".

If grouting is determined to be a potentially effective method of repair, anomaly mitigation may include several of the following steps:

- Coring (when Steel Pipes are used)
- High-pressure cutting of inspection tubes (when PVC Pipe is used)
- High-pressure washing;
- Water flow testing;
- Flushing (high-volume, low-pressure washing);
- Down-hole camera observation;
- Grouting: replacement of permeation:
- Conformance testing,(if deemed necessary) and;
- Final documentation.

The mitigation procedure should be performed by a contractor and crew who are experienced in grouting repair. The mitigation contractor and/or the engineer should keep records during pressure washing, pressure testing and grouting using an approved form / format.

Pressure washing and grouting are typically performed by accessing the anomaly through a cored hole adjacent to a steel inspection tube or through the existing PVC inspection tubes.

Water flow testing is performed after pressure washing and often provides additional information regarding the appropriate mitigation method. Permeation grouting should be performed only if communication with the surrounding geomaterials is evidenced by sufficient flow during water flow testing. If communication is not evident, alternate methods of repair such as replacement grouting may be applicable. Video of the anomaly after pressure washing may also be useful for characterizing the nature of the anomaly and determining the mitigation method.

The grouting procedure is generally intended to increase compressive strength and/or frictional resistance and to reduce the chance of steel corrosion. Grouting generally does not adversely affect geotechnical design criteria. Restoration of earthen materials is typically not required as a result of grouting. Grout and wash water may daylight during repair of anomalies located at relatively shallow depth. Care should be taken so that the surrounding surface soil does not heave. Reduced grout pressures may need to be employed if ground heave is observed.

After completion of the repair procedures, post-mitigation non-destructive testing may be required. Post-mitigation non-destructive testing from the original inspection tubes can be problematic because of the relatively low density of the grout, as well as the difficulty in repairing the anomaly immediately around the tube while preserving the tube's integrity. Coring may be used in some circumstances to recover samples of the grouted materials for visual inspection and strength testing. Typically, the success of grouting mitigation is addressed qualitatively based on observation of the mitigation procedure and review of information such as pressure test results and grout take. The mitigation is typically observed by an engineer, who prepares a mitigation report summarizing the mitigation procedure.

Coring or High-Pressure Cutting of PVC Inspection Tubes

When steel inspection tubes are used (Typically for CSL Testing), a cored hole extending from the top of the pile down to and 6 to 12-inches beyond the anomaly, will be used. Care should be taken when coring in order to use the core to assess the quality or compressive strength of concrete. Typically a double wall core barrel system utilizing a split tube type inner barrel is required. Cores should be logged and labeled for latter use in assessing the repair required. Rotary-drilled holes may be appropriate to provide access for coring.

If additional ports are required beyond an initial cored hole, air-rotary equipment is typically employed. The drilled holes should be at least two inches in diameter. Care should be taken to avoid reinforcing steel. Due to potential difficulties associated with grouting the drilled holes below the anomalous zone, and if the vertical extent of the anomaly is easily detected during trial drilling, drilling typically extends only to the bottom of the anomaly.

When PVC inspection tubes are used, the tubes can be removed from the anomalous zone using a high-pressure washing procedure. The tubes are cut by a stream of high-pressure water, directed laterally against the side of the hole, and rotated as it is slowly withdrawn. Pressure at the pump should be adjusted to account for pressure losses in the line. Excessive pressure loss in the line may

result in pressures at the tip that are insufficient to remove the PVC inspection tubes.

Good results have been obtained using water pressure of 10,000 to 15,000 psi at 10 to 15 gpm. Higher pressure may cut the concrete around the tube, and lower flow rates may be less efficient for removal of the plastic. However, satisfactory results have been achieved with pressure between 7,000 and 30,000 psi. Inspection tube removal should extend from 6 to 12-inches below to 6 to 12-inches above the anomaly. Inspection tubes need not be fully removed at all locations but visual inspection should be performed to approve cleanliness of the area prior to grouting operation.

High-Pressure Washing

The anomaly is pressure washed with high-pressure water concurrently with the tube removal process. The water jet used to cut the tubes also acts on the defective concrete as the plastic is removed. As a general rule, water pressure of 10,000 to 12,000 psi will not affect sound concrete, while higher pressure may remove sound concrete.

A high-pressure submerged jet may be effective at removing segregated concrete, honeycomb concrete and/or inclusions in the concrete. In addition, the jet may break through the concrete to the soil or rock outside of the drilled shaft. The nature of the material cut by the jet can often be determined by observation of the color and grain size of the cuttings returned to the surface. Concrete materials generally run grey to milky with few large pieces, while soil has a wide range of color and heavier suspended solids.

The pressure washing procedure should be monitored to reduce the chance of disturbance of soil adjacent to the shaft while attempting to remove deleterious material from the anomaly. Solids content in the wash return water should be monitored by periodically straining solids out of the effluent. If significant solids content is observed in the return water, the washing may be causing excessive disturbance of the surrounding formation. If native material is not observed in the return water, washing may be continued until the solids content of the return water is deemed insignificant. The contractor should keep a log of communication between holes, water color, type of solids, and estimated solids content.

Water Flow Testing

Water flow testing is performed for each inspection tube. Initially, all other ports are open. Water should be injected through the grout plant, and signs of communication to other holes or to the ground surface should be recorded. The contractor should record the pressure, injection rate and communicating tubes.

After all communicating ports are closed, water flow testing shall be continued to determine whether there is significant communication with the formation. Perform a qualitative evaluation of the water flow test based on injection rate and pressure. A water injection rate into the inspection tube of less than 2 gpm at a pressure of 10 to 20 psi (in addition to the existing hydrostatic pressure in the inspection tube) may be considered insufficient communication for permeation grouting. In the case of insufficient communication, replacement grouting should be considered.

A falling head water flow test may also be used to determine whether there is adequate communication to perform permeation grouting. The criterion for adequate communication is typically more stringent for the falling head test than for the water flow testing described above.

If the flow of groundwater into the inspection tubes is not rapid, the inspection tubes are typically cleared by air injection after water flow testing and prior to down-hole camera observation or grouting.

Flushing

Flushing (high-volume, low-pressure washing) should be performed if there is significant communication between inspection tubes. The purpose of flushing is to remove loose material after pressure washing and prior to grouting or down-hole camera observation. Flushing may involve pumping large amounts of water into an inspection tube and allowing it to return from another or washing material up around a tremmie tube inserted into a single inspection tube. Air, water or alternating injections of air and water may be used for flushing. Solids content in the wash return water should be monitored by periodically straining solids out of the effluent.

Down-Hole Camera Observation

Down-hole camera observation may be performed to verify that the PVC inspection tubes and/or deleterious materials were adequately removed from the anomalous zone. Dry conditions are typically preferable for camera observation. If camera observation is to be performed under water, flushing may be necessary to remove suspended materials from the water within the inspection tubes and scoured anomaly area.

Replacement Grouting

Replacement grouting is intended to fill voids with a relatively low-slump, mortar-type mix. Contaminated concrete and other deleterious material must be removed from an anomaly. This is typically accomplished by high-pressure washing as described earlier, prior to replacement grouting. Flushing may be employed if large

voids are present that allow communication between ports. Grout typically consists of Type I/II or II/V cement mixed at the ratio of one 94-lb sack of cement per 4.5 to 5- gallons of water and is pumped at a pressure of up to 150 psi. Drilling of additional holes into the anomalous zone may be required to promote communication and to reduce the chance of air entrapment within isolated portions of the anomalous zone.

Prior to replacement grouting, the anomaly is cleared of water by injecting compressed air through the inspection tube at the base of the anomaly. Water and air typically return through an adjacent tube, if communication exists, or through the annular space in the tube around the air line.

Permeation Grouting

Typical permeation grouting repair involves high-pressure injection of a water-based, high-slump slurry of cement grout into the pore volume of soil, or contaminated concrete or sediment within a drilled shaft. Permeation grouting results in little disturbance of the material and is accompanied by displacement of pore water. Grain size for microfine cement grout typically ranges from 4 to 8 microns, whereas the grain size for typical Portland cement ranges from 20 to 50 microns. Permeation of the matrix decreases as voids are filled with cement solids.

Permeation grouting is generally effective when the pore aperture or fracture width in soil or rock is approximately five or more times as large as the effective grain size of the grout material. This is often expressed as the groutability ratio ($D_{15,SOIL}$ / $D_{85,GROUT} \geq 5$). However, there are enough exceptions to this rule that a grouting specialist should be consulted before starting a large or costly grouting program. The grouting contractor should typically assess and confirm the feasibility of permeation grouting based on the results of pressure testing.

In most permeation grouting scenarios, there is a pathway for the pore water to escape from the grout injection zone. In the case of grouting a defect in a drilled shaft, there may not be a pathway, or the path may be constricted. This is an inherent limitation on the ability to successfully permeate a concrete defect. Anomaly mitigation by permeation grouting is typically intended to address “soft tip” anomalies, to increase frictional side resistance, and decrease corrosion, rather than to fill all potentially disconnected voids within an anomaly.

In a typical permeation grouting scenario, the grout flow is laminar and displaces most of the trapped pore fluid. If the grout is stable, it cures with less than a 10% separation of solids from the water fraction of the grout. Unstable grout “bleeds”, leaving a layer of solids in the bottom of the pores with a layer of mix water above.

Similarly, a macroscopic void may only be partially filled with solids if unstable grout mixes are used.

The cement solids used for grouting have a specific gravity in the range of 2.75 to 3.17 for ordinary Portland cement. Depending on the water/cement ratio needed to achieve the required viscosity of the grout, density of the grout mix varies but is usually much less than the structural mix used for drilled shaft construction.

The relatively low density of microfine cement may not meet GGL density requirements. When the low-density material in the anomalous zones is permeated or replaced by the grout during the mitigation procedure, GGL post-mitigation non-destructive testing may not result in densities that meet the standard acceptance criterion, which is typically defined as the average surrounding concrete density minus three standard deviations.

Conformance Testing

Quantification of the success of permeation or replacement grouting may require conformance testing. Testing may include Cross-hole Sonic Logging (CSL), Gamma-Gamma Logging (GGL), coring, or excavation. CSL and GGL confirmation testing is often inconclusive due, in part, to the difficulty of washing and grouting adjacent to the inspection tubes while preserving their integrity. The relatively low density of the grout solids also contributes to the difficulty in interpretation of test results. For these reasons, and due to the high cost of invasive confirmation testing procedures, the mitigation procedure is often evaluated qualitatively based on the results of pressure testing, communication and grout take.

Mitigation Report

Upon completion of the mitigation procedure, a post-mitigation report is to be submitted to the Engineer stating what repair work was performed and whether the repair work conformed with the mitigation plan. Any deviations from the mitigation plan must be stated in the report. The Engineer should review the report and determines whether the mitigation efforts were successful.

MITIGATION PLAN "B"

A.1. Coring

1. 3-inch diameter minimum, double wall core barrel system utilizing a split tube type inner barrel is used. This will give a 2 to 2-1/2-inch core sample that can be used for compressive testing.

2. Cored holes should be located as close to the access tube as possible but no further away than 6". Cored holes need to plumb to ensure that they do not cut into the adjacent access tube or into any of the reinforcing steel.

3. At a minimum, cores should extend 6 to 12-inches below the anomalous zone.

4. Cores should be logged and labeled for latter use in assessing the repair required and for compressive strength testing if required.

5. Rotary-drilled holes, drilled to within 24-inches of the anomalous zone, may be appropriate to provide access for coring.

A.2. PVC Inspection Tube Removal

1. The PVC inspection tube shall be cut with high pressure water for the entire elevation range of the anomaly, extending from two feet below the anomaly to two feet above the anomaly. Water jetting shall begin from the lowest anomalous region and proceed upward. Only one anomaly shall be washed and grouted at a time, except where approved in writing by the Engineer.

2. The Contractor shall make provisions to ensure that the required cutting pressure is achieved at the anomaly depth and that the PVC tube is predominately removed at the repair location (complete removal is not necessary). Water pressures typically range from 9,000 to 15,000 psi at a rate of 10 to 15 gpm. Several hundred psi may be lost in the line as a result of pump and line configuration.

B. High-Pressure Washing

1. The anomaly shall be pressure washed with the high pressure water directed laterally against the side of the hole and rotated as it is slowly withdrawn. Water pressure shall be approximately 10,000 psi at 10 to 15 gpm, or as required to remove the deleterious material. Washing shall begin from the lowest anomalous region and proceed upward.

2. Washing will continue until no further solids are observed emanating from the inspection tube and the return flush water is clear, except in the case of erosion of native material, as noted in paragraph 5 below.

3. The Contractor shall monitor the solids content in the wash return water by periodically straining solids out of the effluent.

4. The Contractor shall keep a log of unanticipated communication between holes, water color, type of solids, and estimated solids content.

5. The pressure washing procedure shall be monitored to reduce the chance of disturbance of the formation around the pile while attempting to remove loose sediment and contaminated concrete. Washing shall be discontinued if evidence of significant erosion of native material is observed.

C. Flushing

1. Flushing (high-volume, low-pressure washing) shall be performed if there is significant communication between cored holes or inspection tubes. The purpose of flushing is to remove loose material after pressure washing and prior to grouting or down-hole camera observation.

2. Water shall be pumped into a cored hole or inspection tube and be allowed to return from another hole/tube or around a tremmie tube inserted into a single inspection tube. Air, water or alternating injections of air and water may be used for flushing.

3. Flushing will continue until no significant solids are observed emanating from the cored hole or inspection tube and the return flush water is clear, except in the case of erosion of native material, as noted in paragraph 6 below.

4. The Contractor shall monitor the solids content in the wash return water by periodically straining solids out of the effluent.

5. The Contractor shall keep a log of unanticipated communication between holes, water color, type of solids, and estimated solids content.

6. The flushing procedure shall be monitored to reduce the chance of disturbance of the formation around the pile while attempting to remove loose sediment and contaminated concrete. Flushing shall be discontinued if evidence of significant erosion of native material is observed.

D. Water Flow Test

1. A packer shall be seated in the tube below the top of the concrete, or the inspection tube shall be sealed by other means, as deemed appropriate by the Contractor. The Contractor shall be solely responsible for any health and safety requirements.

2. Valves on all ports shall be open.

3. Water shall be injected through the grout port.

4. The Contractor shall record pressure, injection rate, signs of communication to other ports, signs of communication to the ground surface, amount of water used, color of return water, and time.

5. After all communicating ports are closed, the water flow testing shall be continued to determine whether there is significant permittivity (flow into the formation). A water injection rate into the inspection tube of less than 1 gpm at a pressure of 10 to 20 psi (in addition to the existing hydrostatic pressure in the inspection tube) is typically considered insufficient permittivity for permeation grouting. In the case of insufficient permittivity, replacement grouting is to be utilized, unless other factors provide compelling reasons not to utilize replacement grouting.

6. If permeation grouting is to be performed, the water injection rate will be used to help determine an appropriate water:cement ratio for the starting grout mix. The starting grout mix will be determined based on the attached Chart 1. For example, a take of 20 gpm at 10 to 20 psi

indicates that a thin starting mix (such as mix #1 presented in the attached Grout Mix Table) is preferred. Take of 10 gpm at 10 to 20 psi indicates that a starting mix such as No. 2 or 3 is preferred. Take lower than 10 gpm indicates that lower water:cement ratios are appropriate for the starting mix, as suggested by Chart 1 and determined in the field .

7. If the Contractor suspects insignificant water flow and plans to mitigate by replacement grouting, the falling head test, as described below, may be used as an alternative to the water flow test procedure described in this section. The purpose of the falling head test is to verify that flow is insignificant prior to performing replacement grouting. If the results of the falling head test indicate that flow into the surrounding formation exists, the water flow test described in this section will be performed.

E. Falling Head Test: *To be used in lieu of the "Water Flow Test" when the contractor suspects insignificant water flow into the formation.*

1. If groundwater is within 25 feet (7.6 m) of the top of the inspection tube, the tube shall be extended a minimum of 25 feet above the groundwater table.

2. The inspection tubes shall be filled to the top with water.

3. If communication exists between tubes, the falling head test shall be performed concurrently in communicating tubes.

4. Flow into the formation will be evidenced by a drop in water level inside the inspection tube. If flow into the formation is demonstrated, a water flow test is to be performed. Replacement grouting is to be performed if flow into the formation is not evident.

F. Down-Hole Camera Observation (Optional)

1. Down-hole camera observation shall be performed, if required, after high-pressure washing and flushing. The purpose of down-hole camera observation is to verify that the PVC inspection tubes are predominately removed from the anomalous zone, to verify that deleterious materials have been adequately removed, and to provide additional information regarding the character and extent of the anomaly.

2. Dry conditions are typically preferable for camera observation. If the flow of groundwater into the cored hole or inspection tubes is not rapid, i.e., 2 to 3 gpm at 10 to 20 psi, the core or inspection tubes shall be cleared by air injection after water flow testing and prior to down-hole camera observation or replacement grouting. Camera observation under water may be performed if visibility is acceptable. If camera observation is to be performed under water, flushing will be performed to remove suspended materials from the water within the inspection tubes and scoured anomaly area if visibility is poor.

G. Permeation Grouting

The permeation grouting procedures presented below are intended to serve as the standard procedures for grouting. On occasion, it may be necessary to modify the procedures contained herein in response to field conditions to achieve the desired result. Any alteration of the standard plan should be clearly identified in the submitted post-mitigation report.

a. Evaluate water pressure and rate of take based on water flow test results, as discussed above. If the water flow test indicates that permeation grouting should not be utilized, do not proceed.

b. Permeation grouting requires that sufficient confining pressure be present to conduct grouting operations without grout returning to the surface. Permeation grouting should not be selected for pile mitigation less than 10 feet from the ground (or working) surface.

c. The intent of grouting for drilled shafts is to address the structural, geotechnical and corrosion concerns identified for that foundation element. To that end, grouting purposes to promote the maximum rate of solids injection, as opposed to the maximum rate of grout injection.

d. The Contractor shall be solely responsible for any and all health and safety requirements.

1. Superfine (Nittetsu) cement shall be used for permeation grouting. Grout mix ratios and mix designations are presented in Table 1.
   a. The ratios shown in the attached Table 1 are based on Nittetsu Superfine cement packaged in 22 kg (48.4 lb) bags and having a

specific gravity of 2.75. A batch of permeation grout is typically 33 gallons.

b. Thin grout mixes (such as mixes #1 through #6) are not appropriate for structural mitigation if injected into a void, as they are unstable and will generally not achieve the required design strength.

c. Due to the small grain size of Nittetsu Superfine cement, the mix becomes thixotropic at a water:cement (W:C) ratio of 0.8:1. The superplasticizer also acts as a retarder. Use of superplasticizer will be determined by the grouting Contractor, as required for favorable flow characteristics and to reduce the chance of grouting equipment damage. The Contractor is solely responsible for performing the grouting procedure in such a manner that equipment does not become plugged or otherwise damaged. Use of superplasticizer shall be in accordance with the microfine cement manufacturer's recommendations.

d. The actual volume of the voids is not known, and grout solids are likely to enter the surrounding formation. The Contractor shall secure an adequate supply of cement and water for the repairs. The compressive strength of materials permeated with microfine cement solids depends not only on the strength of the grout, but also on the strength of the solid matrix into which the grout is injected. Strength generation is generally slow due to the superplasticizer. Actual strengths will depend upon grout solids permeation and matrix characteristics.

2.. The starting grout mix will be determined by the grouting contractor based on the results of water flow testing. The starting grout mix will be in accordance with Table 1.

3. Grout shall initially be placed by tremie until it returns from the top of the injection port at a consistency similar to the injected grout. If starting with mix #1 or #2, significant communication and bleed off is anticipated, initial tremie placement is not necessary.

4. Inflate packers to seal tube. Where multiple cores / tubes are being grouted in the same process, all tubes must be grouted simultaneously by means of a common manifold. Alternate means that accomplish the same intent may be utilized where approved by the Engineer.

5. Batches of grout shall be injected under pressure, beginning with the starting grout mix.

6. At the completion of each batch of grout, the Contractor shall evaluate the grout take and pressure to determine the thickness of the next batch of grout to be injected. The grout mix will be increased as pressure increases in general accordance with Table 1. If the starting mix is thicker than the mix indicated by Table 1, continue to use the starting mix.
    a. The grout mix number is increased as the grouting pressure increases, to reduce the chance of premature refusal during a void filling application and to progressively thicken mix to structural mixes as pores within the grouted material become filled.

7. Inject next batch of grout and repeat Step 6 until refusal is reached. For refusal, see Step 11.

8. If the formation does not appear to be plugging (If the pressure does not increase or flow rate decrease after injection under pressure of three full batches), the contractor may elect to thicken the grout by one mix number.
    a. If mix #7 or #8 does not plug the formation quickly, an ordinary Portland cement grout may be used. The replacement-type grout mix shall consist of Type I/II or II/V Portland cement mixed at the ratio of one 94-pound sack of cement per 4.5 to 5- gallons of water.
    b. If plugging does not seem to be occurring with Mix #7, Mix #8 or Portland cement grout, the contractor may shut off the pump for intervals of 2 to 10 minutes to assist grouting process.

9. If grout returns to the surface at any time, note the location and estimate the volume of grout seepage. Also estimate the thickness of the return grout. If the amount of grout return approximates the injection rate, shut off the pump for intervals of 2 to 5 minutes. Use shorter intervals initially or if using thick mixes. In the case of immediate, direct communication, attempt to plug the leak with a half batch of mix #7 followed by a half batch of mix #8.
    a. If grout returns to the surface and interruptions in grouting do not control the leak, the contractor may thicken the grout to the

thickest mix possible and discontinue grouting when the thick grout reaches the surface. Identify this condition in the post-mitigation report.

10. The contractor shall consider known difficulties associated with thick mixes and plan accordingly.
    a. When using thicker mixes such as #7 or #8, check the pressure frequently and be prepared to dilute the mix if signs of plugging in the hoses or fittings are noted (plugging is common with this mix). If the mix is diluted to address plugging of the equipment, do not inject the thinned mix into the pile.
    b. When pumping mix #8, look for signs of hydro-fracturing and test for refusal frequently. (Mix #8 has a very high viscosity and will permeate only a few inches in most geomaterials.)
    c. Mix #8 is often mixed in half-batches, especially for small grouting operations.

11. Refusal is defined as zero take at 150 psi. Grouting pressures shall be held for five minutes prior to release.
    a. Refusal must be achieved with a sufficient quantity of structural grout mix (#7 or #8) for the grouting operation to be considered complete. If refusal is achieved during permeation grouting prior to injection of sufficient quantity of a structural grout mix, mix #8 will be tremied to the bottom of the anomaly location to completely displace thinner mixes.
        i. "Sufficient Quantity" is considered to be greater than or equal to the estimated volume of the cavity developed by high-pressure washing, plus the volume of the tube/grouting port, plus the volume contained in the hoses above the anomaly to the grouting batching plant.
        ii. Filling of the anomalous zone by tremie will be confirmed by consistent return of mix #8 at the top of the tube.
        iii. Upon filling, the grout is to be pressurized to approximately 150 psi and held at that pressure for a minimum of five minutes.
        iv. A sudden pressure drop at high pressures may be a sign that hydro-fracture of the formation has occurred during refusal. Indications of hydro-fracture are to be identified in the post-mitigation report.

12. All Equipment utilized by the Contractor shall be used according to manufacturer's recommendations in a safe manner that will result in the desired finished product.
    a. The grout mixing and pumping unit shall be a colloidal mixer with a progressive cavity injection pump.
    b. Pressure gauges shall be bourdon tubes with 4% accuracy. Gauge protectors shall be used and the gauges shall be replaced on a three- to four-shift cycle. The pressure range of the gauge shall be selected to allow for the anticipated grout pressure to fall in the middle third of the pressure range.

H. Replacement Grouting

1. The cored hole or inspection tube should be completely cleared of water. Extra care is required to assure all water is removed, as residual water will block the grout from completely filling the cavity. Begin tremmie placement in all tubes associated with the anomaly as soon as the water is cleared.

2. The anomaly and cored hole or inspection tube shall be filled with grout by tremmie from the bottom of the voided inspection tube. The tremmie shall be maintained below the level of the grout during placement. The tremmie shall be extracted when the inspection tube is completely filled with grout.
3. After the cored hole or inspection tube is completely filled with grout, the grout shall be pressurized through a port installed in the inspection tube to a minimum of 150 psi and held at that pressure for a minimum of five minutes. The Contractor shall be solely responsible for health and safety.

4. Grout shall consist of Type II cement mixed at the ratio of one sack of cement per five gallons of water. Using a 94-pound sack of cement, the water:cement ratio would be approximately 0.44. The Contractor shall verify that the grout strength corresponds to the required design strength.

5. The Contractor shall have an adequate supply of cement and water for the repairs.

I. Reporting

1. Upon completion of the mitigation procedure, a mitigation report shall be submitted to the Engineer stating what repair work was performed and whether the repair work conformed with the mitigation plan. Any deviations from the mitigation plan shall be stated in the report including explanation of the compelling reason that prompted the modification.

2. The mitigation report shall contain a summary of the repair procedures, which typically includes a summary of observations made during the repair, comparison of the anticipated anomaly volumes with the actual grout quantities used, and the results of testing if performed.

Table 1 – Grout Mix Table						
Mix	Cement (bags)*	Water (gallon)	Weight (lbs)	Volume (gallons)	Density (lbs/gal)	W/C
1	1.0	33.0	319.2	34.9	9.1	6.3
2	2.0	33.0	363.2	36.8	9.9	3.1
3	3.0	33.0	407.2	38.8	10.5	2.1
4	4.0	33.0	451.2	40.7	11.1	1.6
5	5.0	33.0	495.2	42.6	11.6	1.3
6	6.0	33.0	539.2	44.2	12.2	1.0
7	7.0	33.0	583.2	46.0	12.7	0.9
8	8.0	33.0	627.2	48.0	13.1	0.8

Note: * Based on 22 kg (48.4 lb) per bag.

APPENDIX G

ALTERNATIVE MODELS FOR ANALYSIS OF LATERAL LOADING

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APPENDIX G
ALTERNATIVE MODELS FOR ANALYSIS OF LATERAL LOADING

G.1 INTRODUCTION

This Appendix is provided to include an overview of several alternative models reported in the literature for analysis of lateral loading. Alternative models include those based on elastic continuum, boundary element, and finite element models. A brief overview of these models follows, with references for further investigation.

G.1.1 Elastic Continuum and Boundary Element Models

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 while the pinned tip was intended to model a pile bearing on, but not embedded into, rock. While 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. 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 drilled shafts used in practice. Extension of this approach by Carter and Kulhawy (1992) to rigid shafts and shafts of intermediate flexibility, has led to analytical tools for drilled shafts in rock 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).

The continuum models developed by Carter and Kulhawy and by Zhang et al. are described below.

G.1.1.1 Carter and Kulhawy Model for an Elastic Shaft Embedded in an Elastic Rock Mass

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 G-1). The shaft is idealized as a cylindrical elastic inclusion with an effective Young's modulus (E_e), Poisson's ratio (υ_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:

$$E_e = \frac{(EI)_c}{\frac{\pi B^4}{64}} \qquad \text{G-1}$$

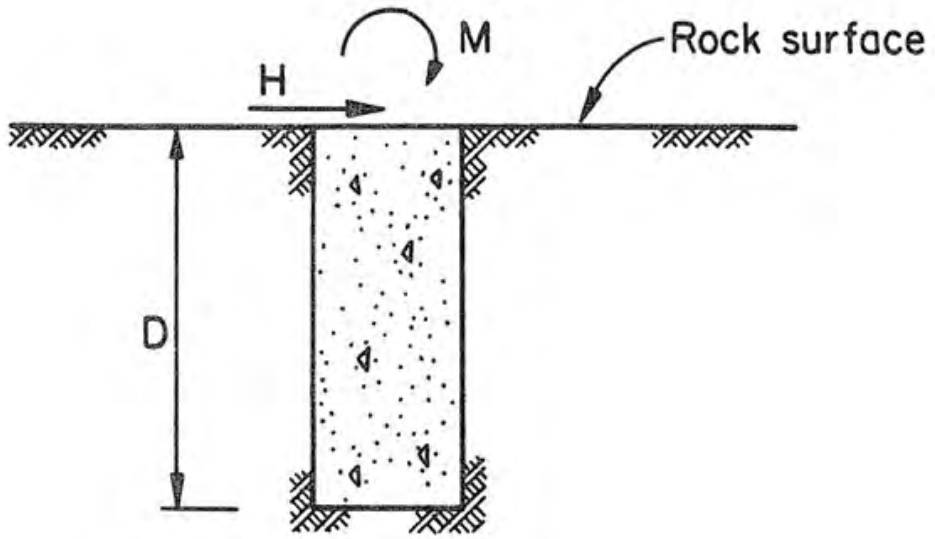


Figure G-1 Lateral Loading of a Rock-Socketed Shaft (Carter and Kulhawy, 1992)

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

$$G^* = G_r\left(1 + \frac{3\nu_r}{4}\right) \qquad \text{G-2}$$

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 ν_r by:

$$G_r = \frac{E_r}{2(1+\nu_r)} \qquad \text{G-3}$$

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:

$$\frac{D}{B} \geq \left(\frac{E_e}{G^*}\right)^{2/7} \qquad \text{G-4}$$

For shafts satisfying Equation G-4, the response depends only on the modulus ratio (E_e/G^*) and Poisson's ratio of the rock mass (ν_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:

$$u = 0.50\left(\frac{H}{G^*B}\right)\left(\frac{E_e}{G^*}\right)^{-1/7} + 1.08\left(\frac{M}{G^*B^2}\right)\left(\frac{E_e}{G^*}\right)^{-3/7} \qquad \text{G-5}$$

$$\theta = 1.08\left(\frac{H}{G^*B^2}\right)\left(\frac{E_e}{G^*}\right)^{-3/7} + 6.40\left(\frac{M}{G^*B^3}\right)\left(\frac{E_e}{G^*}\right)^{-5/7} \qquad \text{G-6}$$

Carter and Kulhawy (1992) report that the accuracy of the above equations is verified for the following ranges of parameters:

$$1 \leq E_e/E_r \leq 10^6 \text{ and } D/B \geq 1.$$

The criterion for a rigid shaft is:

$$\frac{D}{B} \leq 0.05\left(\frac{E_e}{G^*}\right)^{1/2} \qquad \text{G-7}$$

and

$$\frac{E_e/G^*}{(B/2D)^2} \geq 100 \qquad \text{G-8}$$

When Equation G-7 and G-8 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 (ν_r). The following closed-form expressions give reasonably accurate displacements for rigid shafts:

$$u = 0.4\left(\frac{H}{G^*B}\right)\left(\frac{2D}{B}\right)^{-1/3} + 0.3\left(\frac{M}{G^*B^2}\right)\left(\frac{2D}{B}\right)^{-7/8} \qquad \text{G-9}$$

$$\theta = 0.3\left(\frac{H}{G^*B^2}\right)\left(\frac{2D}{B}\right)^{-7/8} + 0.8\left(\frac{M}{G^*B^3}\right)\left(\frac{2D}{B}\right)^{-5/3} \qquad \text{G-10}$$

The accuracy of Equation G-9 and G-10 has been verified for the following ranges of parameters:

$$1 \leq D/B \leq 10 \text{ and } E_e/E_r \geq 1.$$

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

$$0.05\left(\frac{E_e}{G^*}\right)^{1/2} < \frac{D}{B} < \left(\frac{E_e}{G^*}\right)^{2/7} \qquad \text{G-11}$$

For the intermediate case, Carter and Kulhawy suggest 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 G-2. The analysis is approached by structural decomposition of the shaft and its loading, as shown in Figure G-2b. 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.

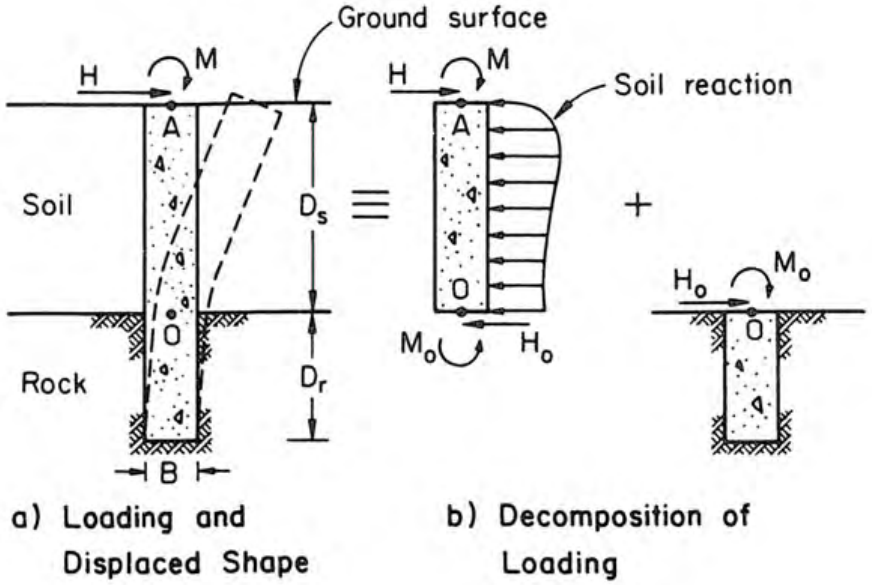


Figure G-2 Rock Socketed Shaft with Overlying Soil Layer (Carter and Kulhawy 1992)

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 (1964), in which the undrained 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 (1964) is assumed:

$$p_u = 3K_p\sigma_v' \qquad \text{G-12}$$

$$K_p = \frac{1+\sin\phi'}{1-\sin\phi'} \qquad \text{G-13}$$

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 G-2. The contribution to groundline displacement and rotation from the loading transmitted to the rock mass (H_o and M_o) is determined based on Equations G-5 and G-6 or Equations G-9 and G-10 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 data base 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) state 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.

G.1.1.2 Zhang, et al. Nonlinear Model for an Elastic Shaft Embedded in an Elastic Rock Mass

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, while 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 response are compared to 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 to 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:

$$H_o = H - \int_0^{z_y} p_{ult} dz \qquad \text{G-14}$$

$$M_o = M + Hz_y - \int_0^{z_y} p_{ult}\left(z_y - z\right)dz \qquad \text{G-15}$$

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 G-2. 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 ($\phi = 0$) and fully-drained loading ($c = 0$). For fine-grained soils (clay), undrained loading conditions are assumed and the limit pressure is given by:

$$p_{ult} = N_p c_u B \qquad \text{G-16}$$

$$N_p = 3 + \frac{\gamma'}{c_u} z + \frac{J}{2R} z \le 9 \qquad \text{G-17}$$

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

$$p_{ult} = K_p^2 \gamma' z B \qquad \text{G-18}$$

where K_p = Rankine coefficient of passive earth pressure defined by Equation G-11. Ultimate resistance of the rock mass is given by:

$$p_{ult} = (p_L + \tau_{\max})B \qquad \text{G-19}$$

where τ_{max} = maximum shearing resistance along the sides of the shaft 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 Geological Strength Index (GSI). The resulting expression is:

$$p_L = \gamma' z + q_u \left(m_b \frac{\gamma' z}{q_u} + s \right)^a \qquad \text{G-20}$$

According to Zhang et al. (2000), a computer program was written to execute the above procedure. Predictions using the proposed method are compared to 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.

G.1.2 **Finite Element Soil Models**

Software now exists that will permit the nonlinear analysis of drilled shafts or groups of drilled shafts using the finite element method (FEM) with relative ease on a high-end PC or a workstation, for example ABAQUS (Hibbett et al., 1996). FEM analysis is justified when the soil or rock conditions, foundation geometry or loading of the group is unusual. An example of a case in which a comprehensive FEM analysis might be conducted is for designing a group of drilled shafts that are to be socketed into sloping rock on a steep mountainside, in which it is necessary to use permanent tiebacks to secure the drilled shaft group to stable rock.

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APPENDIX H

PRELIMINARY GUIDANCE FOR POST-GROUTED DRILLED SHAFTS

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APPENDIX H
PRELIMINARY GUIDANCE FOR POST-GROUTED DRILLED SHAFTS

H.1 INTRODUCTION

While drilled shafts have occasionally been post-grouted for more than four decades, the practice of post-grouting of drilled shafts has become much more common in the U.S. in the past decade. Practices for design and construction of post-grouted drilled shafts are currently evolving based on collective experience, and recent and ongoing research. As such, this appendix is intended to provide preliminary guidance related to post-grouted drilled shafts, based largely on a recent state-of-the-practice report developed by a team of researchers from the International Association of Foundation Drilling (ADSC) for FHWA (Loehr, et al, 2017) and a workshop sponsored by FHWA, Caltrans, ADSC, and the Deep Foundations Institute (DFI) on quality assurance for post-grouted drilled shafts (Large, 2016). Interested readers are encouraged to review the FHWA report for more detailed information regarding current practices and procedures for post-grouted drilled shafts, and for additional information related to the mechanisms for improvement, alternative design approaches, and recommended practices related to execution and QA/QC for post-grouted drilled shafts. FHWA guidance related to post-grouted drilled shafts is likely to change in coming years, so readers are additionally encouraged to seek out future guidance documents when embarking on projects where post-grouting will be considered or used. While post-grouting has been used to improve both side and tip resistance for drilled shafts, post-grouting with the specific intention of enhancing side resistance is uncommon in North American practice. The content of this appendix, therefore, exclusively addresses post-grouting at the tip of drilled shafts.

H.2 POST-GROUTING PROCESS AND MECHANISMS FOR IMPROVEMENT

As considered in this appendix, post-grouting involves installation of a grout delivery system at the tip of a drilled shaft during assembly of the reinforcing cage. After constructing the shaft as normal and allowing the concrete to achieve adequate strength, grout is injected through the device at relatively high pressure as illustrated schematically in Figure H-1. The grout pressure produces a bi-directional load at the shaft tip that simultaneously pushes upward on the drilled shaft and downward on the soil/rock beneath the shaft tip. The upward load is resisted by the weight of the drilled shaft and downward side resistance mobilized along the length of the shaft, while the downward load is resisted by tip resistance mobilized beneath the shaft tip. Following grouting, the mobilized side and tip resistance may be subject to some relaxation, but some load nevertheless remains in the shaft. This “pre-mobilized load” serves as the starting point for subsequent top-down loading of the shaft from the superstructure.

Loehr, et al. (FHWA, 2017) found that post-grouting can improve the performance of drilled shafts founded in all types of ground conditions, through two mechanisms termed “pre-mobilization” and “ground improvement”. The pre-mobilization mechanism refers to the beneficial influence of the pre-mobilized load induced in the shaft as a result of post-grouting. Pre-mobilization effectively “stiffens” the load-settlement response of the shaft to allow greater shaft resistance to be mobilized for a given settlement. Pre-mobilization alone does not increase the ultimate shaft resistance but will frequently increase the nominal design resistance because greater resistance can be mobilized at settlements corresponding to some selected failure criterion. In contrast, the ground improvement mechanism refers to beneficial improvements to the ground beneath the shaft tip that increase the magnitude of available tip resistance, whether those changes are produced by densification of ground beneath the shaft tip, by permeation of grout into the ground beneath the tip, or by enlargement of the shaft tip or some

combination of these effects. Unlike pre-mobilization, ground improvement will increase the ultimate resistance for a post-grouted shaft compared to that for a comparable ungrouted shaft, which will also generally increase the nominal design resistance. Pre-mobilization is a purely "mechanical" phenomenon that results from reversal of loading, and is not predicated on having any specific material characteristics. In contrast, ground improvement is highly dependent on material characteristics and, thus, will vary substantially depending on the type of material present at the shaft tip. Ground improvement is likely to be more substantial for loose granular materials, potentially producing up to a 40 percent increase in tip resistance, but quite limited for cohesive soils or rock. Since pre-mobilization is purely a mechanical phenomenon, it is also likely to be more consistent and reliable than improved performance due to ground improvement. However, improvement due to pre-mobilization is subject to uncertainty associated with the load that can be developed during grouting, which depends on the grout pressure and the area over which this pressure acts, as well as relaxation that may occur following grouting.

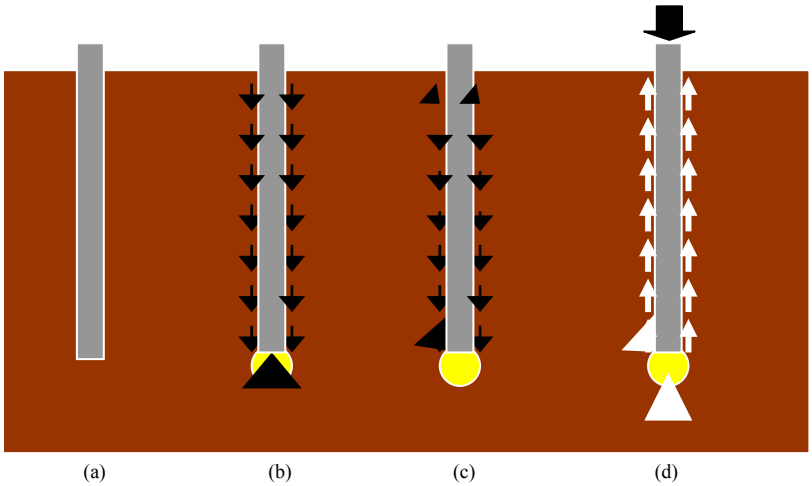


Figure H-1 Post-grouting Process: (a) shaft constructed; (b) grout pressure at tip, reacts against side shear; (c) some relaxation occurs, but some preload remains; (d) structural loads on shaft engage side and tip resistance

Post-grouting should not be considered as a means to remedy poor shaft construction and base cleaning operations. However, post-grouting does provide the benefit of compressing any debris left from imperfect cleaning of the shaft base and, to some extent, restoring disturbance to the natural materials beneath the shaft tip due to excavation methods.

H.3 GROUTING PROCEDURES AND QA/QC

Post-grouting devices generally fall into two broad categories: flat jack devices and sleeve-port (or tube-a-manchette) devices. Grouting devices are typically mounted at the bottom of the reinforcing cage as shown in the photos of Figure 3-16, with tubes or hoses tied to the cage that extend to the surface for subsequent use. Flat jack devices usually consist of grout delivery tubes connected to a steel plate with a rubber membrane wrapped underneath, and are typically limited to smaller diameter drilled shafts. Sleeve port devices typically consist of two to four grout pipe circuits, or "U-tubes," arranged in various configurations at the shaft tip. The U-tubes are perforated for grout release and covered by tight fitting rubber sleeves to prevent ingress of concrete during placement of the shaft concrete. A layer of gravel, typically less than 18 inches thick, may sometimes be placed beneath the grouting device to provide a level surface for the device and to facilitate distribution of grout across the base of the shaft.

Grouting is typically performed after the concrete has achieved a compressive strength of at least 2,500 psi. Grouting lines are generally first flushed with water until water exits the return lines at the top of the shaft to ensure communication through the tubes and verify that the grout has a flow path. Water is also used to pressurize the system and break the rubber sleeves in sleeve-port systems to ensure that outflow is achieved below the tip of the drilled shaft. Grout is then batched and pumped with the return lines open to purge the system of water. When grout is observed to flow from the return line(s), valves on the return line(s) are closed to pressurize the system and begin grouting. Grout used for post-grouting is normally a "neat" water-cement mixture with a water/cement ratio ranging from 0.40 to 0.55, although higher water/cement ratios are sometimes used at the beginning of grouting. Type I or II Portland cement is mixed with water in a colloidal mixer in order to fully mix the grout; paddle mixers should not be used because they may not produce the uniform colloidal mixture of cement and water needed for effective post-grouting. Grouting pressures for post-grouting depend on the specific application and ground conditions, but may approach 800 psi or more in some applications; grouting equipment lines should therefore be capable of such high pressures.

Normal grouting procedures consist of pumping the grout at a steady flow rate while recording grout pressure, the volume of grout delivered, and uplift of the top of the shaft, as illustrated in Figure H-2. In some cases, strain gauges may be installed near the tip of the drilled shaft to monitor load in the shaft during grouting. Figure H-3 illustrates the desired response during post-grouting based on measured grout pressure, grout volume, and shaft uplift, plotted with respect to one another. As shown in the figure, the desired response to grouting is gradual and typically proportional to increases in grout pressure, grout volume, and shaft uplift as indicated by measurements progressing along diagonal lines in the three graphs. Deviation from the desired proportional response generally indicates that a grouting limit has been achieved, as described subsequently, or that some problem is occurring (e.g., blocked grout lines) so that grouting is no longer effective. Grout pressure is generally monitored with a pressure transducer attached to the grout pump. Grout volume is generally monitored by measuring the change in the level of grout in the mixing tank. Shaft displacement is generally monitored using displacement transducers or dial gauges attached to a reference beam as illustrated in Figure H-2.



Figure H-2 Grout pump used for post-grouting (left) and instrumentation to monitor grouting operation (right).

Grouting is generally continued until one of three limiting criteria are observed:

1. The target grout pressure is achieved, after delivering a sufficient quantity of grout to fill the theoretical volume of the grouting device.
2. Upward shaft movement exceeds some established threshold. Upward shaft movement has commonly been limited to 1/2 inch or less. However, greater movement can likely be tolerated in many geomaterials without significantly compromising side resistance. Additionally, upward movement of the shaft is a strong indication that grout pressures are inducing substantial load and mobilizing downward side resistance; thus, upward movement of the shaft is, in fact, a desirable outcome of post-grouting.
3. The volume of grout delivered exceeds some specified maximum grout volume threshold, especially if pressures are observed to stabilize with additional grout "take." In general, if grout pressure is not proportionally rising with delivered grout volume, additional grout delivery will not achieve more improvement. A sharp increase in grout volume under uniform grout pressure or with a dropping pressure may be an indication of hydraulic fracturing of the ground; this should be avoided and grouting terminated if this is evidenced in the monitoring data.

Grouting termination criteria are generally site and project specific. As such, appropriate values for each criterion should be carefully considered as part of design for post-grouted drilled shafts.

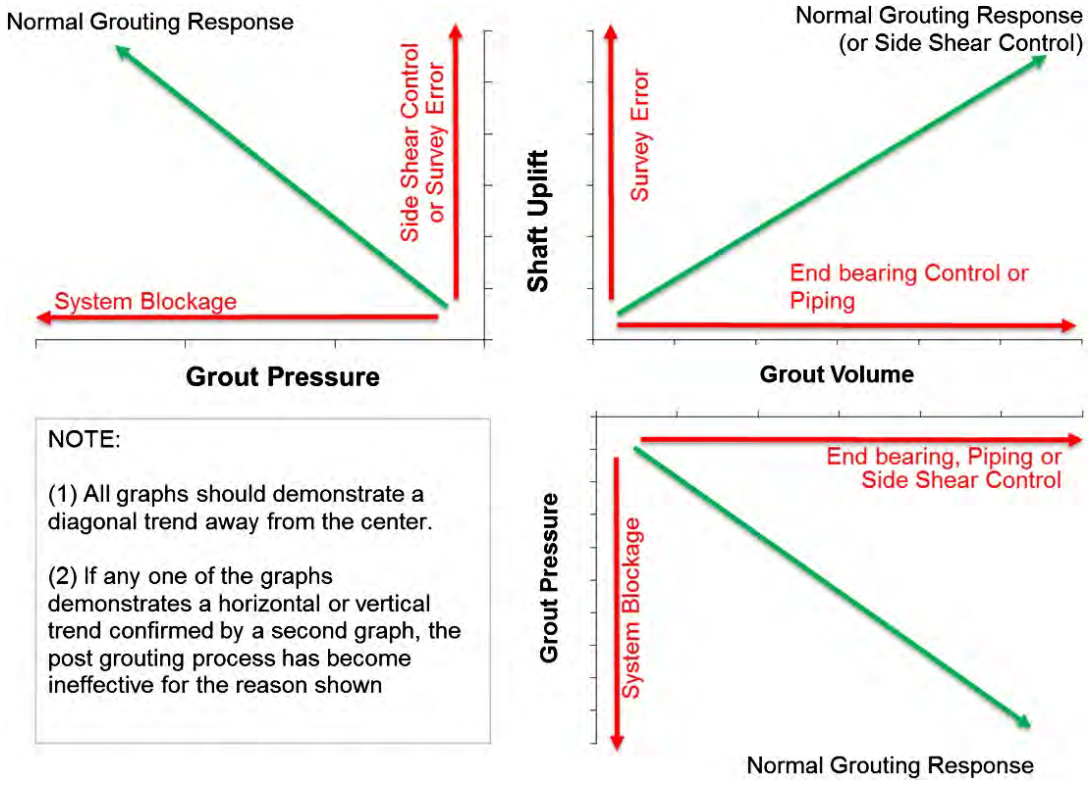


Figure H-3 Graphical representation of grouting parameters to control post-grouting operations (Mullins, 2015).

H.4 DESIGN FOR POST-GROUTED DRILLED SHAFTS

Design for post-grouted drilled shafts should include estimating the nominal axial resistance according to some desired failure criterion as well as establishing appropriate values for grouting termination criteria. Several quasi-empirical methods have been proposed for predicting the nominal axial resistance for post-grouted drilled shafts (e.g., Ruiz, et al., 2005; Mullins, et al., 2006; Dapp and Brown, 2010; Dai, et al., 2011). Each of these methods utilize some form of "multipliers" that are applied to estimates of nominal resistance for an ungrouted shaft to establish estimates for the nominal resistance for post-grouted drilled shafts. Evaluations performed for the FHWA study (Loehr, et al., 2017) indicate the accuracy of these design methods is highly variable. The FHWA report therefore recommends that the nominal axial resistance for post-grouted drilled shafts be estimated using load transfer (i.e., *t-z*) methods that allow for separately considering pre-mobilization and ground improvement mechanisms. Additional details regarding such methods are provided in the FHWA report. Load tests should also generally be performed to confirm the resistance provided by post-grouted drilled shafts using project specific construction and grouting procedures.

In addition to estimating the nominal axial resistance, designs for post-grouted drilled shafts should include appropriate grouting termination criteria that will serve as the basis for QA/QC in the field. These criteria include establishing a target grout pressure and minimum grout volume, a maximum grout volume, and a maximum shaft uplift. The minimum grout volume should be established from the volume of grout required to fill the grout tubes and grouting device, with some allowance for the available precision for measuring grout volume. The maximum grout volume should be established in proportion to the shaft base area to reflect a condition where loss of grout containment may be indicated (e.g., one or two feet multiplied by the shaft area). Upward shaft movement is often limited to approximately 1/2 inch, although greater movement can often be tolerated.

The target grout pressure should be carefully considered and established from evaluation of the achievable grout pressure for the site specific conditions present. A common practice with early projects involving post-grouting has been to establish the target grout pressure based on the pressure need to achieve a given nominal resistance, without explicitly considering whether such pressure can actually be achieved in the field. This practice introduces substantial risk for construction and can lead to costly claims and remedial measures. It is therefore important that rational estimates be made for the grout pressure that can be achieved given the ground conditions present at a site. In general, the grout pressure that can be achieved during post-grouting is limited by the pressure that will cause the shaft to lift out of the ground, the pressure that will exceed the unit tip resistance of the ground beneath the shaft tip, the pressure that will cause hydrofracture of the soil at the shaft tip, and the limitations of the grouting equipment used. The selected target grout pressure should be based on the least pressure that will produce any of these effects, with due consideration of the potential variability among different shafts.

Designs for post-grouted drilled shafts should also address steps that will be taken if the target grout pressure cannot be achieved during construction. In cases where the target grout pressure is not achieved during the initial grouting attempt, it is often possible to achieve greater grout pressures by temporarily halting grouting operations for some period of time (typically a few hours) prior to attempting a second phase of grouting. While such "staged grouting" will often allow greater grout pressures to be achieved, the practice should be avoided because the grout pressure may not be applied over the entire area of the shaft tip, and thus may not induce the intended loading in the shaft that is responsible for much of the improved performance compared to ungrouted shafts.

H.5 SUMMARY

Post-grouting of drilled shafts is a means to improve the performance of drilled shafts. Current post-grouting practices in the U.S. utilize flat-jack or sleeve port devices installed at the tip of drilled shafts to deliver neat cement grouts to the shaft tip. The delivered grout produces a bi-directional load at the shaft tip, which produces improvement in the load-settlement response of the shaft, and may improve the ground at and below the shaft tip to further improve performance. Designs for post-grouted shafts should be developed using load-transfer methods to predict the magnitude of improvement that can be expected from post-grouting, and should explicitly consider the magnitude of grout pressures that can be realistically achieved for the ground conditions present.